STABILIZATION OF EASTERN SAUDI SOILS USING HEAVY FUEL OIL FLY ASH AND CEMENT KILN DUST

BY

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A Thesis Presented to the
DEANSHIP OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS
DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL ENGINEERING

MAY, 2009

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS
DHAHRAN 31261, SAUDI ARABIA

DEANSHIP OF GRADUATE STUDIES

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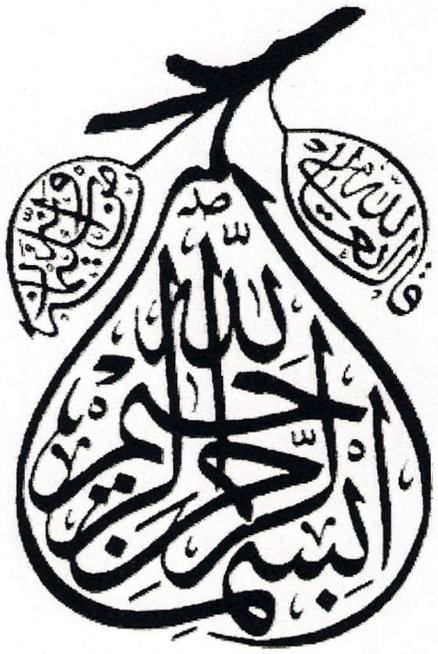

In the Name of Allah, Most Gracious, Most Merciful.

إهداء

أهدى هذا العمل المتواضع:

إلى من أمرت ببرهما وطاعتهما

إلى من أسال الله تعالى أن يوفقنى لبرهما ورضاهما

إلى من أدين للفضل بعد الله لهما

إلى والدى العزيزين

إلى إخوانى وأخواتى الاعزاء

إلى زوجتى و أبنائى الأحباء

إلى كل من له فضل على من الأحبة والأصدقاء

أهديكم هذا الجهد المبارك وأسال الله تعالى أن يوفقنى وإياكم لجنات الخلد وأن يرزقنا سعادة الدنيا والآخرة إنه ولى ذلك والقادر عليه.

ACKNOWLEDGMENT

All praises and thanks be to ALLAH (subhana wa taala), the Almighty, whose blessing and help are all the time with me, and He is the only one Who helps and His help is best for all.

Acknowledgement is due to King Fahd University of Petroleum and Minerals for the support given to this research through its tremendous facilities.

I acknowledge, with deep gratitude and appreciation, the encouragement, inspiration, valuable time and guidance given to me by my thesis advisor Prof. Omar S. Baghabra Al-Amoudi throughout this research. Thanks are extended to my co-advisor Prof. Saad A. Aiban for his valuable guidance and help. I would also like to express my sincere gratitude and thanks to my other committee members, Prof. Sahel N. Abduljauwad, Prof. Muhammad H. Al-Malack and Dr. Muhammad Kalimur-Rahman, for their constructive guidance and support during this work.

I would like to extend my thanks to Dr. Mohammed Essa and Engineer Imran for their continuous help during the execution of my experimental program. I would also like to acknowledge Al-Derbas Company which provided me the marl soil, the Arabian Cement Company Limited (ACCL), Jeddah, that provided the cement kiln dust, and Mr. Abdullah Mohammed Al-Hadhrami who provided the heavy fuel fly ash from Al-Shuaibah Power Plant, Jeddah.

Finally, I extend my thanks to my mother, father and all my family members for their patience, emotional and moral support throughout this study and my academic career and also for their love, encouragement and prayers.

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ABSTRACT

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Title: Stabilization of Eastern Saudi Soils Using Heavy Fuel Oil Fly Ash and Cement Kiln Dust

Major Field: Civil Engineering (Geotechnical)

Date of Degree: May, 2009

In the Kingdom of Saudi Arabia, it is anticipated that an increasing amount of oil fuel fly ash (FFA) will be produced from power plants firing crude and heavy fuel oils. Therefore, power plants in the Kingdom may face difficulties concerning the disposal of the produced FFA. Similarly, there are several companies of cement manufacturing all over Saudi Arabia producing thousands of tons of cement daily. These companies face a problem of disposing a large quantity of their cement kiln dust (CKD), which is considered as a waste material. Hence, it would be a noble task if these waste materials are utilized in civil engineering projects. Therefore, this study was conducted with the principal objective of investigating the potential usage of these two waste materials in the stabilization of two indigenous soils (i.e. sand and non-plastic marl).

To achieve this objective, the two types of soil were treated with different dosages of FFA and CKD. The mixtures of these stabilized soils were thoroughly evaluated using compaction, CBR, unconfined compression and durability tests. The results of these tests were analyzed and the effect of CKD and FFA on the engineering properties of these mixtures was optimized and compared with the traditionally used stabilizing agent (Portland cement).

Master of Science Degree

Department of Civil Engineering

King Fahd University of Petroleum and Minerals

Dhahran, Saudi Arabia

الخلاصة

الإسم: جميل مهيوب سيف عبدالله

العنوان: تثبيت التربة في المنطقة الشرقيه بالمملكة العربية السعودية بإستخدام رماد النفط الثقيل المتطاير وغبار أفران الأسمنت

التخصص: هندسة مدنية (جيوتقنية)

التاريخ: مايو، 2009

نظرا للزيادة المتوقعة فى كميات رماد الوقود النفطى المتطاير (FFA) فى المملكة العربية السعودية والناتجة من محطات الطاقة التى تستخدم النفط الثقيل كوقود. ولذلك، فإن هذة المحطات تواجه صعوبات تتعلق بالتخلص من الكميات الهائلة من الرماد النفطى المتطاير (FFA) والذى يعتبر كمخلفات يجب التخلص منها. وبالمثل، هناك العديد من مصانع الاسمنت المنتشرة فى جميع أنحاء المملكة العربية السعودية والتى تنتج الآف الأطنان من الأسمنت يوميا. تواجه هذة المصانع صعوبات تتعلق بالتخلص من الكميات الهائلة من غبار أفران الأسمنت (CKD)؛ الناتج من صناعة الأسمنت؛ والذى يعتبر كمخلفات يجب التخلص منها. لذلك يعتبر إستخدام هذة المخلفات (FFA، CKD) فى مشاريع الهندسة المدنية من الأهداف البيئية والأعمال النبيلة. ولهذا، فإن هذة الدراسة تعتمد على هدف أساسى هو إستكشاف ودراسة إمكانية إستخدام هذة المخلفات فى تثبيت تربتين طبيعيتين هما الرمل والتربة الجيرية المعروفة محليا ب "المارل".

ولتحقيق هذا الهدف؛ تم خلط ومعالجة هاتين التربتين بكميات مختلفة من الرماد النفطى المتطاير (FFA) وغبار أفران الأسمنت (CKD) وتم تقييم الخلطات الناتجة من التربتين المعالجتين بإجراء عدة إختبارات مثل الدمك؛ نسبة تحمل كاليفورنيا (CBR)؛ قياس القوة الانضغاطية غير المحصورة والتحمل. وقد تم تحليل نتائج هذة الإختبارات ودراسة تاثير كلا من غبار الأسمنت (CKD) والرماد النفطى المتطاير (FFA) على الخواص الهندسية ومقارنتها بالخلطات المعالجة باستخدام الاسمنت.

درجة ماجستير

قسم الهندسة المدنية

جامعة الملك فهد للبترول والمعادن

الظهران، المملكة العربية السعودية

Chapter 1

INTRODUCTION

It is well known that the rapidly growing population and expansion of industrial facilities in Saudi Arabia are increasing the demand on electric utilities. As power demand grows by 7 percent or more each year, the Saudi Ministry of Electricity and Water estimates that the country will require up to 20 gigawatts (GW) of additional power generating capacity by 2019 [http://www.middleeastelectricity.com]. In addition to the electric power, Saudi Arabia needs to double its available resources of drinking water as the population will almost double to 40 millions by the year 2020. Saudi Arabia is investing heavily in increasing the power and drinking water capacity. Shuaibah is the first power and water project in Saudi Arabia, and the first of a total of four planned major projects. The goal of these projects is to increase the power plant capacity by 4,500 MW and to provide an additional 2.2 million cubic meters of drinking water daily.

Saudi Arabia is utilizing gas for power generation utilities as part of the government's plans to expand gas utilization; however, it is also known that the bigger power plants in Saudi Arabia are fueled by oil. This may be less favorable than gas from environmental point of view. However, it will not have a high environmental impact on global warming and greenhouse gas accumulation as compared with the usage of coal to produce power, which is used by most industrialized nations like USA, India, China, etc., where other options are also available like oil and nuclear power. Therefore, future plans witness a large increase in the use of oil as fuel for power plants.

Fuel oil is not widely used in power plants in other parts of the world, partly because of fluctuation in oil prices, however, in Saudi Arabia which has the largest proven reserves of oil in the world, it is a readily available and economically feasible to use fuel for such purposes [Dincer and Al-Rashed 2002]. Just like coal, which is also used for electric power generation in many countries, the process of power generation produces huge quantities of fly ash as a solid waste. The amount of fly ash produced and its physical characteristics, as a powder, would create a real problem to manage.

A review of the literature indicates a lot of research being undertaken to find ways and means of reusing the fly ash produced from burning coal in power plants. However, the fly ash produced from fuel oil is not widely investigated, although it is totally different in many of its characteristics and chemical composition from the coal fly ash. Its contents of hydrocarbons, heavy metals, sulfur, and residue ash are different. Hence, its impact on the environment is different and its uses and ways of disposal are also different. Therefore, research studies are needed to explore ways and means of utilizing the heavy fuel oil fly ash and its safe disposal, particularly for the Kingdom of Saudi Arabia, which produces large quantities of this type of fly ash.

Similarly, there are many cement factories all over the Kingdom of Saudi Arabia producing thousands of tons of cement daily. Some of these factories face a problem of producing large quantities of cement kiln dust (CKD). CKD is produced as a by-product in the Portland cement manufacturing process. For example, the Arabian Cement Company Ltd. (ACCL), Jeddah, produces about 1000 tons of CKD per day, a minor portion of which is recycled into the kiln and a small portion is being used by contractors while the rest is disposed of in landfills. Due to its high levels of chlorides and alkalis,

many cement manufacturers are reluctant to recycle such CKD into the production lines [Kessler, 1995; USEPA, 1998]. There is, however, a potential for reusing CKD in several fields due to its high lime content and cementitious properties. ACCL currently produces about 1000 tons per day of CKD, which is expected to double after the completion of its expansion project.

CKD has certain disadvantages, which make it difficult for reuse and recycling. Large dosages of CKD in concrete are difficult due to the high chloride and alkali contents. Further, the considerable fineness of the material makes handling, transport and storage of CKD very difficult. In spite of these disadvantages, considerable research is being undertaken on CKD around the globe to find ways and means to economically use it on industrial scales rather than disposing it in the landfills [El-Sayed et al., 1991; Salem et al., 2001; Batis et al., 2002; Maslehuddin et al., 2009]. CKD has certain characteristics which can be exploited to generate economical usage in several areas including concrete, pavements, soil stabilization, and wastewater treatment [Bhatty, 1995] in addition to the success in recycling a significant proportion of the CKD into the kilns.

In this study, the usage of both CKD and FFA in the stabilization of indigenous soils (non-plastic marl and sand) collected from the Eastern Province of the Kingdom of Saudi Arabia was investigated.

1.1 Significance of This Research

Since heavy fuel oil fly ash (FFA) and cement kiln dust (CKD) are considered as waste materials, it would be a noble task if these waste materials are being utilized in civil engineering applications such as the stabilization of indigenous soils. There are four types of soil in the Eastern Province of Saudi Arabia, namely, clay, sabkha, marl and

sand. Sabkha and clay are problematic soils and their usage in construction projects is very limited. Therefore, this research was intended to investigate the possibility of incorporating CKD and FFA in the stabilization of two selected indigenous eastern Saudi soils, namely, non-plastic marl and sand.

1.2 Objectives

The main objective of this study was to investigate the possibility of utilizing cement kiln dust (CKD) and fuel oil fly ash (FFA) in the stabilization of two selected indigenous eastern Saudi soils. The primary objectives of this investigation are the following:

- To select and characterize two selected eastern Saudi soils (non-plastic marl and sand).
- To stabilize the two selected soils with CKD and FFA. The potential type and dosage of the stabilizing agent would, thereafter, be selected on the basis of the results of the maximum strength (unconfined compressive strength and California bearing ratio) and durability assessment (ASTM D 559 and Slake Durability Test).

To achieve these objectives, two different types of soil, namely, non-plastic marl and sand, from the Eastern Province, were treated with different dosages of CKD and FFA. The mixtures of these stabilized soils were evaluated using compaction, CBR, unconfined compression and durability tests. Based on the results of these tests, the optimum dosage of CKD and FFA for each of the two soils was identified.

Chapter 2

LITERATURE REVIEW

2.1 Heavy Fuel Oil Fly Ash

Coal and heavy fuel oils are primarily used in industrial and utility boilers to generate steam that is used in heat and electricity generation. Coal use is more common in most countries than is heavy fuel oil. Over 90% of coal is consumed by utilities in the generation of electricity, with a much smaller amount of electricity being generated from the combustion of heavy fuel oil. Heavy fuel oil is also used in utility and industrial plants and to a smaller extent in the transportation sector, almost entirely in ocean-going ships. Heavy oil use largely occurs in areas where the fuel can be transported by ships or barges or where there are local oil resources that can be used near the production sites [U.S. Department of Energy (2000a), (2000b) and (2000c)].

Fly ash is defined as the finely divided residue that results from the combustion of coal or oil-fired power generators. The generated fly ash consists of very small individual particles that are carried up and out of the boiler with the flow of exhaust gases leaving the boiler after the coal and/or oil is consumed. The combustion process may also result in the enrichment of trace elements in the ash, most often adhered to the surface of the ash particles. The quantity and characteristics of fly ash depend primarily on the fuel characteristics and the burning process [NCASI, 2003]. The fly ash generated from combustion of fuel oil is the unburned residue found in the fuel and the additives used. It contains the organometallics from crude oil, inorganic contaminants or metallic catalyst fines used in the refining process. In addition to carbon, major elements in fuel oil fly ash

include magnesium, vanadium, nickel and sulfur. On the other hand, coal fly ash particles are enriched in arsenic and selenium. Heavy fuel oil ash and coal ash particles have unique elemental compositions which distinguish them from each other.

Although the fundamental particle formation processes during the combustion of heavy fuel oils are the same as those for pulverized coal, there are distinct differences between the two fuels. In contrast to coals, oils do not typically contain significant extraneous or include mineral matter. The metals in heavy fuel oils are generally inherently bound within the organic molecule, which may be the case for only a small portion of the metals in higher rank coals. Unlike coal, interactions between volatile metal species and nonvolatile minerals within the heavy fuel oil droplets are much less likely in heavy fuel oils.

Hersh et al. [1979], Piper and Nazimowitz [1985], and Walsh et al. [1991] showed that, in contrast to pulverized coal, the majority of the sampled fly ash masses from residual fuel oil combustion in power plants is likely to lie below 1.0 μm in diameter, although larger particles can form with poor carbon burnout. Furthermore, Walsh et al. [1991] have demonstrated that Fe, Mg, and Ni are concentrated at the center of the submicron particles, while Na and V are associated with a "halo of sulfate residue." Bacci et al. [1983] found substantial enrichment of both Ni and V in the submicron particle size fraction of samples collected at a large oil-fired power plant.

2.1.1 Fly Ash Characteristics

Fly ash is a powdery residue generated by the power stations that use heavy oil as the source of fuel. As shown in Tables 2.1 and 2.2, fly ash contains relatively high heavy metal content, particularly vanadium (as V_2O_5) and nickel (as NiO). In addition, the

residual carbon level in the fly ash is very high. Typical fuel oils contain Fe, Ni, V, and Zn, in addition to aluminum (Al), calcium (Ca), magnesium (Mg), silicon (Si), and sodium (Na). Transition metals [iron (Fe), manganese (Mn), and cobalt (Co)] and alkaline-earth metals [barium (Ba), calcium (Ca), and magnesium (Mg)] may also be added for the suppression of soot or for corrosion control [Bulewicz et al. 1974; Feldman 1982].

The chemical characteristics of the fuel oil fly ash generated at a power plant differ significantly from that of coal fly ash. The carbon content of heavy fuel oil fly ash is about 95% while that of coal flyash generally ranges between 20% and 50%. Toxic heavy metals, such as vanadium (2.08% as V_2O_5) and nickel (0.37% as NiO) are also present in the heavy fuel oil fly ash. The high carbon content and presence of toxic heavy metals suggested that this fuel oil fly ash be considered as a hazardous respirable dust that demands careful handling and safe disposal to ensure proper environmental protection.

2.1.2 Ash Disposal Alternatives

Fly ash is produced in large quantities as a by-product of the combustion of coal, gas, crude and fuel oil in power plants. It is collected from flue gases mainly through air pollution control devices. A very small fraction of the collected coal fly ash is utilized currently as a supplementary cementing material for cement, concrete industries, and other purposes. The remaining large fraction adds to the major waste disposal problem for the industries [Yazıcı, 2007]. A number of disposal alternatives are practiced for different types of wastes. The disposal alternatives that are most commonly used for fly ashes are briefly discussed below.

2.1.2.1 Landfills

The main goal of landfill or burial method is to dispose wastes in an environmentally safer manner that ensures a minimum migration of hazardous components through soil, water and air. The basis of landfill design is simply to contain the ash waste rather than to treat it to some useful products. The ash disposal sites could be unlined or lined depending mainly on the chemical composition of the ash, and the nature of the disposal site. Due to the increasing concerns about groundwater contamination, the use of liners for any landfill is becoming more common. In fact, liners are one of the most important design elements of hazardous waste landfills that control leachate, the liquid that is composed of infiltrating water and liquid waste components [Watts, 1997].

2.1.2.2 Wet Settling Basins

Wet basins have been historically the most widely used method for ash disposal due to their relatively low cost and simplicity of operation. Where topography is suitable and space is available, the use of lagoons is a standard practice to dispose huge amount of fly ash. Lagoons are ponds or lakes typically located near the power plants. In USA, the majority of lagoons located around the country have no liners or groundwater monitoring option despite the concentrated levels of heavy metals and many other contaminants [http: //www.hecweb.org/ccw/CCWdoc.html].

Ash from boilers and collectors is carried by a hose system to the disposal area. The ash settles in the lagoon. The overflow, which is usually alkaline, is discharged into the nearest watercourse. With properly designed lagoons and appropriate skimmer devices to prevent discharge of any floating ash, this system of ash disposal from power

plants is satisfactory, particularly from the stand point of stream and groundwater pollution control [http: //www.hecweb.org/ccw/CCWdoc.html].

Table 2-1: Physico-Chemical Properties of Heavy Oil Fly Ash (Kwon et al., 2005)

Parameter	**Value**
Moisture Content (%)	11.54
Bulk Density (g/cm^3)	0.52
True Density (g/cm^3)	2.15
Porosity (%)	10.31
Sulfur (%)	3.26
Carbon (%)	76.13
Oxygen (%)	1.92
Nitrogen (%)	1.24
Residual Ash (%)	19.85

Table 2-2: Heavy Fuel Oil Ash Analysis (Daous, 2004)

Parameter	Quantity
PH @ 18°C	2.8
Moisture	0.33 wt %
Unburned Carbon @ 700° C	90.18 wt %
Ash Content	9.82 wt %
SO_3	3.06 wt %
Vanadium as V	4007 ppm
Nickel as Ni	1021 ppm
Iron as Fe	559.4 ppm
Magnesium as Mg	1800 ppm

2.1.2.3 Incineration

Another possible alterative method for ash disposal is to burn fly ash in incinerators with the help of auxiliary fuel. The carbon content of fly ash generated at power plants that burn heavy fuel oil is significant (i.e. above 90%). The high carbon content and the volume of such fly ashes could be reduced significantly after incineration. The residual ash obtained after incineration could be rich in heavy metal contents that could encourage recovery of heavy metals. However, because of the presence of toxic metals in fuel oil fly ash, there exists a high possibility of emission of harmful gaseous pollutants and particulate matter into the atmosphere during improper incineration of the fly ash [Seggiani et al. 2007]. Relevant literature on this subject is currently not available. Therefore, this alternative method should be carefully investigated, particularly in terms of potential environmental pollution from toxic emission before its application.

2.1.3 Potential Reuses/Recycling of Fly Ash

Potential uses of fly ash as a resource material for different purposes have been explored by various research agencies, scientists and institutes [Dermatas and Meng, 2003; Prabakar et al., 2004; Sezer et al., 2006; Yazici, 2007]. Most of the studies have addressed fly ash generated from burning coals. Literature on reuse and/or recycling of fly ash generated from combustion of heavy fuel oil (HFO) is very scarce because of the limited use of heavy fuel oil for power generation. Therefore, specific research programs should be initiated to identify possible uses for fuel oil fly ashes.

Experience indicates that coal fly ash has a potential for reuse, particularly in the following civil engineering and related applications [Dermatas and Meng, 2003]:

- As an admixture in concrete (due to its pozzolanic nature).

- As raw material for cement manufacturing.
- Highway construction.
- Slope stabilization.
- Waste management.
- Agriculture.

Most of the fly ash reuses that are reported in the literature are related to ash generated from coal. Other potential uses include re-burning for full utilization of the energy of unburned carbon in the fly ash. The proper reuse and/or recycling of fly ash is desired over the conventional disposal practices not only for its economic benefits but also for ecological advantages. Therefore, fly ash generated from combustion of heavy fuel oil, which has not been significantly used for beneficial applications similar to those reported for other fossil fuel ashes [EPA. 1999], should be studied for consideration of reuse.

2.2 Cement Kiln Dust (CKD)

As the raw feed travels through the Portland cement kiln system, particulates of the raw materials, partially processed feed and components of the final product, are entrained in the combustion gases flowing countercurrent to the feed. These particulates and combustion gas precipitates are collected in the particulate matter control device (e.g., cyclone, baghouse, or electrostatic precipitator), are collectively referred to as cement kiln dust (CKD).

Generation of CKD is estimated at approximately 30 million tons worldwide per year [Dyer et al., 1999; Maslehuddin et al., 2008]. Large quantities of CKD are produced during the manufacture of cement clinker by the dry process. While modern dust-

collecting equipment is designed to capture virtually all CKD and much of this material can today be returned to the kiln, for various reasons, a significant portion, in some cases as much as 30 to 50% of the captured dust, must be removed as industrial waste [Kessler, 1995; USEPA, 1998]. As a result, in the United States, more than 4 million tons of CKD, unsuitable for recycling in the cement manufacturing process, require disposal annually [Todres et al., 1992]. Recently, the IEEE-IAS Cement Industry Committee and PCA provided a summary of management practices of CKD in period 1990 to 2006, as shown in Table 2.3. It can be seen that the annual use of CKD for beneficial applications has ranged from as low as 574,800 metric tons to 1.16 million metric tons and the quantity of CKD landfilled decreased dramatically when compared to the quantity of clinker produced. It dropped from 60 kg/metric ton in 1990 to 16 kg/metric ton in 2006.

CKD contains a mixture of raw feed as well as calcined materials with some volatile salts. It is derived from the same raw materials as Portland cement but, as the CKD fraction has not been fully burnt, it differs chemically from the former. Typical analyses for UK cements are given in Table 2.4 [Aidan and Trevor, 1995]. The chemical composition may, however, vary with the type of the raw materials and the cement manufacturing process.

Taha et al. [2004] evaluated the possibility of recycling the waste materials in the Sultanate of Oman. In particular the usefulness of copper slag (CS) and CKD were investigated for use as partial replacements for Portland cement in mortar mixtures. The data developed in that study indicated that CKD would perform better than CS in concrete when utilized as a partial Portland cement replacement. Also, the use of CKD as an activating agent with CS would enhance quite significantly the compressive strength

of cement mortars. Among all mixes, the mix containing 5% CKD + 95% OPC yielded the highest 90 days compressive strength of 41.7 MPa in comparison with 40 MPa for the mix containing 1.5% CKD + 13.5% CS + 85% OPC.

CKD is considered a valuable material that is currently labeled as waste but can be used in many applications including the following [Bhatty, 1995]:

- Agriculture: potash/lime source and animal feed.
- Civil engineering: fill, soil stabilization, fly ash stabilization and blacktop filler.
- Building materials: lightweight aggregates, blocks, low strength concrete and masonry cement.
- Sewage and water treatment: coagulation aid and sludge stabilization.
- Pollution control: sulfur absorbent, waste treatment and solidification.

Maslehuddin et al. [2008] studied the properties of cement-CKD combination. Results indicated that CKD did not adversely affect the properties of cement mortar and can be used without affecting the requirements stipulated by ASTM C 150 for Portland cement. Also, results indicated that early age and 28-day compressive strength of CKD cement mortar is higher than that of Type I cement mortar and the shrinkage of CKD-cement mortar increases with an increase in the quantity of CKD.

El-Sayed et al. [1991], Al-Harthy et al. [2003], and Maslehuddin et al. [2008] investigated the effect of CKD on the compressive strength of cement paste and on the corrosion behavior of embedded reinforcement. The studies reported that up to 5% substitution of CKD by weight of cement had no adverse effect on cement paste strength and on the reinforcement passivity.

Table 2-3: Historical Cement Kiln Dust Production and Management*

Year	Plants responding to survey for given year	CKD beneficially reused on or off site, metric tons	CKD sent to landfill, metric tons	CKD reclaimed from landfilled, metric tons	Annual clinker production, metric tons	CKD sent to a landfill/clinker produced, kilograms / metric tons
1990	84	752,152	2,655,725	No data	44,360,364	60
1995	94	651,205	3,146,952	No data	61,729,315	51
1998	95	768,601	2,499,651	13,409	67,104,547	37
2000	92	574,803	2,223,190	79,171	68,263,086	33
2001	102	924,552	2,329,132	231,904	75,683,170	31
2002	101	664,848	1,989,680	103,223	77,636,598	26
2003	102	718,410	1,995,143	116,416	79,356,511	25
2004	102	917,968	1,993,421	69,099	83,945,430	24
2005	102	987,717	1,429,150	104,952	85,568,243	17
2006	101	1,160,011	1,403,062	261,351	86,686,834	16

Note* – From PCA member company surveys

Table 2-4: Typical Chemical Composition of CKD and Portland Cement [Aidan and Trevor, 1995].

Constituent	CKD, %	Ordinary Portland Cement, %
SiO_2	11-16	22
Al_2O_3	3-6	5
Fe_2O_3	1-4	3
CaO	38-50	64
MgO	0-2	1
SO_3	4-18	3
K_2O	3-13	< 1
Na_2O	0-2	< 1
Cl	0-5	< 0.1
Loss on ignition	5-25	1
Free CaO	1-10	2

A similar conclusion was reached in an investigation carried by Batis et al. [2002] where it was found that when CKD and blast furnace slag are added in proper ratio in

ordinary Portland cement, the compressive strength and corrosion resistance of the mix increases. Salem et al. [2001] have studied the hydration of cement pastes containing granulated slag and CKD made with and without silica fume. It was reported in their study that the hydraulic reactivity of granulated slag and silica fume, as activated by raw CKD, is relatively high as compared with those activated by washed CKD. The reason has been attributed to the presence of excess alkali contents in raw CKD. Because of its high total lime content, CKD can also be used in-lieu of lime for soil stabilization.

2.2.1 Review of Research on Usage of CKD in Soil Stabilization

2.2.1.1 Introduction

In the field of geotechnical engineering in general and soil stabilization in particular, the parent soils are practically categorized under either cohesionless soils (i.e., sandy and other coarse-grained soils) or cohesive soils (i.e., primarily clay and silt). In the context of this literature, this categorization is valid because currently lime, sometimes in combination with fly ash and possibly with some Portland cement addition, is considered as the premium material for clay soil stabilization [Sreekrishnavilasam et al. 2007]. Since the soil stabilization mechanism requires calcium as the major stabilizing agent, it is possible that some CKDs, especially those high in free lime, would similarly be useful in stabilizing clay soils. In the case of sandy soils, which are commonly selected in the pavement layers, the usage of CKD may provide cementitious materials when it is mixed with water in a way similar to the mechanism by which Portland cements provide their binding characteristics [Al-Amoudi et al., 2006; Al-Aghbari and Dutta 2008]. Further, the fine CKD particles tend to fill the pore void in the sand matrix thereby producing a

compact and dense structure [Freer-Hewish et al., 1999]. Therefore, these approaches could potentially open up a very large market for cement plants that waste nearly 5 million tons of CKD in North America alone every year [Bhatty et al., 1996]. Therefore, this review is presented as a guide for selecting and using CKD to stabilize indigenous soils.

It is noteworthy to mention that the available information on the use of CKD for such applications is preliminary isolated and lacks quantitative data, as most of the work has been done only on selected soils and selected CKDs. It has been suggested that in order to have an insight on the stabilization potential of CKD and a complete understanding of the underlying mechanism, comprehensive and systematic studies on CKD-soil stabilization are needed. This would require a selection of CKDs from different plant operations, and a selection of sub grade soils and expansive clays [Peethamparan et al., 2008]. The effect of CKD on the engineering properties needs to be optimized and compared with traditionally used stabilizing agents such as hydrated lime, fly ash, and Portland cement.

The use of CKD as a stabilizer of marginal soils for subbase and base applications could potentially consume a bulk of the CKD being wasted every year. Such a use would enhance the engineering characteristics of unsuitable and marginal soils, allowing their use for improved subgrade, subbase or related applications, with the additional benefits of reducing both solid waste and the exploitation of scarce and dwindling natural resources [Baghdadi and Rahman 1990; Freer-Hewish et al. 1999; Al-Aghbari and Dutta 2008]. Currently, hydrated lime is used as the major stabilizer in the stabilization of clays, and Portland cement for granular and low plasticity materials. Combinations of lime and fly

ash (with or without the addition of Portland cement) are also sometimes used [Sezer et al. 2006].

2.2.1.2 Mechanisms of Soil Stabilization

There are two basic mechanisms of stabilization which operate in stabilizing sandy soils and clayey soils. Sandy materials, being volumetrically stable and less plastic, are strengthened (as measured by the unconfined compressive strength, bearing capacity, etc.) by direct cementitious effects of Portland cement. The pozzolanic reactions with or without additives, such as fly ash, is normally provided by either Portland cement, lime, CKD, or even the additive itself when the free lime content is high. Such a mechanism is termed pozzolanic stabilization. The main reaction in this case takes place between the reactive silica in the soil itself or fly ash, and $Ca(OH)_2$ from cement, lime, or CKD, to form calcium silicate hydrate. The pozzolanic reactivity is greatly increased with the concentration of available alkali in solution [Helmuth, 1987].

On the other hand, clay soils having high volume instability, extreme sensitivity of bearing strength to moisture content, and high plasticity, undergo stabilization via ion-exchange mechanisms mediated by calcium-containing additives such as lime, hydrated lime [$Ca(OH)_2$], CKD, and Portland cement. Such additions promote cation-exchange primarily by exchanging the sodium ions on the cleavage surfaces of the clay minerals with calcium ions and causing flocculation/agglomeration of particles, resulting in granular materials of low plasticity, low sensitivity to moisture fluctuation with respect to volume change and bearing capacity, etc. Such a mechanism may be termed ion-exchange stabilization.

It might be noted, however, that such stabilization would be dependent upon the cation exchange capacity (CEC) of the clay soil in question and the availability of calcium ions from the calcium-containing additive, such as CKD. The CEC of clay is dependent upon its composition. For instance, a clay soil containing montmorillonite usually has a CEC ranging from 80 to 150 milliequivalent (meq) per 100 gram as compared with 3 to 15 meq for kaolinite and 10 to 40 meq for illite clays [Christensen, 1969]. One has to be aware, however, that the CEC of a clay may be affected by the interference of soluble alkalies (e.g. salts of K and Na), that can be readily released from CKDs. The amount and nature of the exchangeable ions in a clay also affect the properties of soils. For instance, calcium-saturated clays are usually more friable than sodium-saturated clays. Consequently, the workability of soils can be improved by replacing the sodium ions with calcium ions [Christensen, 1969].

2.2.1.3 CKD Characteristics for Soil Stabilization

Any potential application of CKD, including sand and clay stabilization, is governed by the physical and chemical composition of the dust [Peethamparan et al. 2008]. In practical terms, the dusts vary markedly from plant to plant in chemical, mineralogical, and physical composition, depending upon the feed raw materials, type of kiln operation, dust collection facility, and the fuel used [Klemm, 1980]. In general, CKDs are particulate mixtures of partially-calcined and un-reacted raw feed, clinker dust, and fuel ash, enriched with alkali sulfates, halides, and other volatiles. Haynes and Kramer [1982] have reported an approximate phase composition of CKD as shown in Table 2.5.

Table 2-5: Approximate Composition of a Cement Kiln Dust [Haynes and Kramer, 1982].

Constituent	% by weight	Constituent	% by weight
$CaCO_3$	55.5	Fe_2O_3	2.1
SiO_2	13.6	KCI	1.4
CaO	8.1	MgO	1.3
K_2SO_4	5.9	Na_2SO_4	1.3

The CKDs generated from long-wet and long-dry kilns are generally composed of partially calcined kiln feed fines enriched with alkali sulfates and chlorides. The dusts collected from the alkali bypass of precalciner kilns tend to be coarser, more calcined, and concentrated with alkali volatiles. Dusts from gas- or oil-fired kilns contain higher proportions of soluble alkalies (in the form of K_2SO_4) as compared to coal-fired kilns. However, such a generalization is not valid in many cases [Todres et al., 1992].

The use of CKD in soil stabilization, sewage treatment, etc., primarily depends upon its physical and chemical characteristics, most importantly, the lime content and the fineness of particles. The source of calcium in CKD could be present in the form of $CaCO_3$ and free CaO. It may also be present as $Ca(OH)_2$ resulting from partial hydration with spray water applied to control dust emission, or from atmospheric moisture during open stockpiling. A high loss on ignition (LOI) in the CKD may imply that the CKD contains a large amount of $CaCO_3$ and/or that it has been exposed to moisture [Peethamparan et al. 2008].

When CKD is exposed to moisture, alkali sulfates rapidly go into solution. Free lime and some cementitious phases, if present, undergo hydration. As a result, the availability of calcium ions is dictated by the equilibrium achieved through the solubility limit of

$Ca(OH)_2$ and gypsum ($CaSO_4.2H_2O$) if present. Evaporation of moisture from open stockpile will result in the formation of gypsum and alkali hydroxides. Each of these compounds will contribute to higher LOI, however, LOI of a CKD not exposed to moisture may be related to the $CaCO_3$ content that undergoes decarbonation when ignited.

The availability of calcium ions from CaO, $CaCO_3$, and $Ca(OH)_2$ differs because of variations in their molecular structures and relative solubilities in water. Boynton [1980] indicated that the release of calcium ions from $CaCO_3$ is the least compared to CaO and $Ca(OH)_2$. Therefore, CKDs containing $CaCO_3$ will provide less calcium ions and take longer time to promote soil stabilization as compared to the CKDs containing CaO or $Ca(OH)_2$. Fine particles provide large specific surface which can promote a more rapid reaction. In the case of soil stabilization, the source of calcium ions may be less important. Rather, their availability for ion-exchange, and heat of hydration due to the reaction below, may be of some importance in the rate of strength development.

$$CaO + H_2O = Ca(OH)_2 + \triangle H \text{ (exothermic heat)}$$

Stabilization with $Ca(OH)_2$ is often preferred in field conditions as it is less corrosive than CaO (as it lacks exothermic reaction with water) and thus safer to handle.

2.2.1.4 Use of CKD for Stabilization of Sandy Soils

Napeierala [1983] examined the possibility of using CKD in stabilizing sandy soils for pavement subgrade applications. It was reported that an addition of 15% CKD having 5.9% free CaO and MgO, and 0.97% total alkalies (K_2O + Na_2O) ensured a compressive strength of 360 psi (2.5 MPa), which is a standard practice in Poland for the

subgrade within 14 days of the treatment. Accordingly, the kiln dust was considered a viable substitute for cement or lime for stabilizing the subsoils.

Baghdadi and Rahman [1990] studied the effects of CKD on stabilizing western Saudi siliceous dune sand in highway construction. The geotechnical properties of the various mix proportions, such as the 7-day unconfined compressive strengths, moisture-density relationship, CBR, swelling, and the economic considerations, were investigated. It was deduced that a mix proportion of 30% CKD and 70% sand gave peak performance for application as base materials. In a somewhat similar study conducted later, Baghdadi et al. [1995] reported that the use of CKD between 12 and 50% was satisfactory to stabilize dune sand. For light application, 12 to 30% CKD was found sufficient, and for heavily-loaded application, about 50% CKD gave satisfactory stabilization. In general, CKD-stabilized dune sand exhibited increased compressive strengths with increased CKD addition and curing durations. However, higher than 50% CKD gave poor durability properties.

Freer-Hewish et al. [1999] found that wind blown sand can be stabilized using CKD for use in road pavement structures, however, large amounts of CKD were needed to meet pavement layer standards. Therefore, they studied the effect of adding chemical additives in reducing CKD requirement and achieving the same bearing at the same time.

Field experience on subgrade modification/stabilization with CKD is at this time still quite limited. The Oklahoma DOT performed a field evaluation of CKD treated subgrades in 2000 (Miller et al., 2003) The results of this investigation, which involved treatment of a sandy lean clay (PI ~ 15-30%) with three different CKDs, confirmed laboratory observations (the significant improvement in properties that can be obtained

using CKD, but also the great variability in stabilization results depending on the type of CKD used), and highlighted a number of issues relevant to construction (e.g. the problem posed by wind blown CKD). Additional field work, as well as laboratory tests with different CKDs (LOI ranging between 22 and 29%) on shale-sand mixtures (Miller et al., 2003), showed that CKD can perform better than free lime, and indicated better performance for the lower LOI CKD.

Al-Amoudi et al. [2006] investigated the stabilization of four eastern Saudi soils using CKD. The addition of CKD to the different types of soil, namely sandy sabkha, white marl with low plasticity, cohesionless marl and plastic marl, resulted in a decrease of the dry density and increase in the optimum moisture content. The unconfined compressive strength exhibited substantial increase of about 5.66, 1.69, 1.41 and 13.2 times by the addition of 50% CKD to the sandy sabkha, white marl with low plasticity, cohesionless marl and plastic marl soils, respectively. Similarly, Shabel [2006] studied the stabilization of Jizan sabkha soil using cement and cement kiln dust (CKD). He concluded that the addition of CKD stabilizer to the Jizan sabkha soil improved its engineering properties.

Al-Aghbari and Dutta [2008] investigated the effect of cement and cement by-pass dust on the engineering properties of sand. They found that sand with ordinary Portland cement can be a good material for base or subbase course application whereas the sand with cement by-pass dust can be used for improving the bearing capacity of sand to support low to moderate rise building.

2.2.1.5 Effect of CKD on Kaolinite and Bentonite Stabilization

Baghdadi [1990] reported the usage of CKD for stabilizing pure kaolinite and a 50:50 kaolinite-bentonite clay mixtures. Pure bentonite clay was highly plastic (PI ≈ 520), whereas the kaolinite was less plastic (PI ≈ 9). The 50:50 kaolinite-bentonite clay mixture gave a PI of 150. The following tests were carried out to characterize the stabilized mixtures: Plasticity indices, moisture-density relationship, unconfined compressive strength and durability. Results of this study indicated the following conclusions [Baghdadi, 1990]**:**

1. **Plasticity Indices:** Addition of CKD significantly reduced the plasticity of kaolinite-bentonite mixture and bentonite clay. It was found that 8% CKD addition was required (as compared to 2% lime) to reduce the plastic index of pure bentonite from 520 to 340 in seven days. Addition of 8% CKD (or 2% lime) reduced the PI of the 50:50 kaolinite-bentonite mixture from 150 to 120.

2. **Moisture-Density Relationship:** Standard Proctor tests on kaolinite-treated CKD mixtures showed that with increasing CKD additions (from 0-100% by wt.), maximum dry densities of the material increased from 1.43 to 1.54 g/cm^3 while the optimum moisture content decreased from 28 to 23%. High density (2.75 g/cm^3) and fine particle size range (6 to 100 μm) of CKD might be the reason for the increased dry densities of the compacted material.

3. **Unconfined Compressive Strength:** The addition of CKD significantly improved the unconfined strength of pure kaolinite. Increasing additions resulted in increased strengths at longer curing ages. A 7-day strength of 1,460 kPa was

obtained with 30% CKD additions, which exceeded the maximum strength (1,380 kPa) required for soil cements in bases and subbases applications. According to the ACI Committee 230 Report (ACI, 1990), the minimum 7-day q_u specified for subbase and subgrade in rigid pavement construction by the USA Army Corps of Engineers (USACE) is 1380 kPa (200 psi) and for base course 3450 kPa (500 psi). For flexible pavement construction, however, these values are 1725 kPa (250 psi) and 5175 kPa (750 psi), respectively.

Addition of 4% CKD also improved the strengths of kaolinite-cement and kaolinite-lime mixtures. The effects were more pronounced with the kaolinite-cement mixtures than the kaolinite-lime ones.

4. **Wetting-Drying Durability:** As per the wet-dry durability test procedure [PCA, 1971], the weight losses of 8, 6, and 4% were recorded at 15, 30 and 50% CKD-kaolinite mixtures, respectively. According to Bhatty et al [1996], the weight loss of soil cement in the wet-dry durability tests should not exceed 10%. This suggests that in this particular case where kaolinite is the starting material, the addition of 30% CKD can satisfy both strength and durability requirements for soil applications in bases and subbases, which is highly demanding.

Miller and Azad, [2000] found that an increases in the unconfined compressive strength (q_u) of soil occurred with the addition of CKD. Increases in q_u were inversely proportional to the plasticity index (PI) of the untreated soil. Significant PI reductions occurred with CKD treatment, particularly for high PI soils.

Miller and Zaman [2000], based on laboratory and field test data, indicated that CKD was more effective than the quicklime for soil stabilization. They found that CKD-soil

stabilization can be cost effective and that it required less type of soil construction time than the treatment with quicklime.

Peethamparan et al. (2006) performed an experimental study to investigate the effectiveness of CKD for stabilizing kaolinite clay. The two CKDs used have free lime content of 13.85 and 5.32% and LOI of 14.22 and 29.63%, respectively. The percentage CKD varied from 8 to 25% by dry weight of clay. They reported that the strength of CKD-treated kaolinite clay is proportional to the CKD content and also to the free lime content. For example, for a CKD with 13.85% free lime and LOI of 14.22%, considerable improvement in the strength of kaolinite is observed (e.g. for 15% CKD, the 7-day q_u increased 6 times). Also, for a CKD with higher free lime content (13.85%), the increase in compressive strength at 7 days is twice that of the CKD with lower free lime (5.32%).

2.2.1.6 Use of CKD in Expansive Clays

A study on the use of CKD in clay stabilization was also reported by Zaman et al. [1992] and Sayah [1993]. They established potentially useful correlations among the engineering properties of the clays and their stabilized counterparts. However, their investigations were based on only one CKD and primarily one clay, a dark grey "fat" clay, although, at times, some selected tests were also carried out on other potentially expansive clays. The CKD contained a fairly high fraction of uncalcined $CaCO_3$ (high LOI) and low alkalies. The primary clay used in the investigations belonged to the CH group [Spangler and Handy, 1992]. The clay-CKD mixtures containing 5% to 40% CKD by weight were cured for up to 56 days. The results showed that, with the exception of the dry densities, the engineering properties of the CKD-clay mixtures were comparable to those of fly ash-soil and cement-soil mixtures.

Overall, the results documented in the literature indicate that, due to the wide variation in the physical and chemical properties of CKD, general conclusions on its validity as a soil stabilizer cannot be easily drawn. Moreover, the investigations conducted so far have been limited to comparative experimental investigations and the mechanisms responsible for the improved behavior remain unclear. To date, these factors have essentially prevented a more extensive use of CKD in soil stabilization.

One of the most noticeable phenomena that are mentioned in almost every article on lime, fly ash or CKD stabilization is the ability of the binder to change the plasticity characteristics of the soils. It has been shown by many researchers that the addition of CKD to moderately plastic to highly plastic soils generally causes an immediate increase in plastic limit and reduction in plasticity index [McCoy and Kriner, 1971; Baghdadi, 1990, Zaman et.al., 1992; Miller and Azad, 2000]. However, note that in the literature, trends of both increasing and decreasing liquid limit with CKD percentage are reported depending on the soil used [Zaman et.al., 1992; Miller and Azad, 2000].

2.3 Use of CKD and Fly Ash for Soil Stabilization

Most of the reported literature indicates that CKDs have been used in the field of soil stabilization with fly ash. This conjoint usage of CKD and fly ash is ascribed to the fact that both materials are waste components of the cement industry and energy generation utilities, respectively. Both materials are produced in millions of tons annually and, therefore, their incorporation in the field of soil stabilization becomes of great potential to the developed countries in the world. However, the situation in Saudi Arabia is different because coal fly ash is not produced since energy is generated through

the use of oil and gas, which produce fuel fly ash (FFA). Literature in the field of using FFA in soils stabilization is scarce.

Nicholson presented a series of patents [1977, and 1982] for a series of investigations on CKD and fly ash mixtures for producing subbase materials with different aggregates. CKD was used up to 16% by weight of the mixture, producing a durable mass by reacting with water at ambient temperatures. It was claimed that the stabilized mixtures acquired strengths and other performance characteristics comparable to those of cement-aggregate or lime-fly ash-aggregate bases. It was also pointed out that these materials required less energy to produce and cost less than the traditionally-used asphalt aggregate bases that required heating.

Collins and Emery [1983] demonstrated the effectiveness of substituting CKD for lime in a number of lime-fly ash-sandy aggregate systems for subbase construction. The results indicated that the majority of the CKD-treated fly ash and aggregate mixtures resulted in materials which were comparable in strength (in many cases exhibited even better early strengths), durability, dimensional stability, and other engineering properties, to those of the conventional lime-fly ash-aggregate mixtures. They also indicated that both the chemical and physical compositions of the CKDs were important in controlling the reactivity and the resulting engineering properties of the dust-fly ash-aggregate mixtures. A combination of free lime, MgO, alkalies, and a favorable particle fineness (fraction between 20 and 70 μm) almost always enhanced the reactivity of CKD and produced high compressive strengths, whereas CKDs with higher LOI and low free lime impeded the reactivity and gave lower strengths.

Miller et al. [1980] reported the use of CKD and fly ash as the cementitious ingredients in developing pozzolanic bases that demonstrated comparable properties to those of a stabilized base. These observations were based on a series of laboratory and field tests on a number of laboratory-prepared samples. These samples also possessed the property of autogenous healing, by which the hairline cracks formed in the mixture are healed because of a continued chemical reactivity. This property of CKD could be brought to beneficial application as it would enable the mixes to steadily gain strength even after developing hairline cracks due to shrinkage stresses exceeding the tensile strength at a given time. It was pointed out, however, that the use of any particular CKD-fly ash combination would require an appraisal of chemical and strength test data to establish optimum properties for a suitable mix design.

Some research has indicated that, if the fly ash (FA) and CKD are appropriately blended, the alkalis from CKD may activate the hydration of FA and the blends may create a cementitious material in which the waste material deficiencies will be converted into benefits [Wang et al., 2004]. Bhatty [1984, 1985 and 1986] studied binary, ternary, and quaternary mixes using ordinary Portland cement (OPC), five different CKDs, two different types of FA (Class F and C), and slag. He observed that cements containing CKD alone had reduced strength, setting time, and workability. The addition of FA into a CKD-OPC system lowered the alkali content and resulted in improved strength. Dyer et al. [1999] examined ternary blends containing two types of CKD, pulverized fuel ash (PFA), and OPC, and he found that CKD accelerated the binder hydration.

Daous [2004] studied the utilization of CKD and FFA in cement blends in Saudi Arabia. CKD produced in a local cement production plant along with fly ash resulting

from combustion of heavy fuel oil in a local power generation plant were utilized as waste material blends with Portland cement, produced from the local plant, at various proportions. He reported that satisfactory mechanical strength (a minimum of 94% of compressive strength of ordinary Portland cement) can still be achieved in blends utilizing 90% cement and not more than 4% fly ash. Adequate mechanical strengths (a minimum of 80% of compressive strength of Portland cement) were achieved in blends utilizing as little as 70% cement when only kiln dust was blended.

Chapter 3

EXPERIMENTAL PROGRAM

The main objective of this study is to investigate the potential usage of cement kiln dust (CKD) and fuel fly ash (FFA) in soil stabilization. To achieve this objective, two different types of soil, namely sand and non-plastic marl, from the Eastern Province of Saudi Arabia, were treated with different dosages of FFA and CKD. This chapter outlines the procedures and tests methods which were followed to fulfill this objective. The experimental program consisted of six phases, as shown in Fig. 3.1. The first phase was to select and collect the soils from the Eastern Province. In the second phase, these soils were characterized in the laboratory using ASTM standards. Chemical stabilization of the selected soils using CKD and FFA was pursued in the third phase. Based on the results obtained in the third phase, either the stabilizer was rejected in the fourth phase or a detailed stabilization programs on the non-plastic marl or sand soils were pursued in the fifth phase. In the sixth phase, based on the strength, durability and economy requirements, selection of type and dosage of stabilizer was proposed. All phases of the experimental program are discussed thoroughly in the following sections.

3.1 Collection of Soils

Two Eastern Saudi soils, non-plastic marl and sand, samples were collected from different places. The non-plastic marl sample was obtained through Al-Derbas Company which collected it from the area located along the Dhahran-Abqaiq highway in the Eastern Province while the sand sample was collected from Dhahran dune sands.

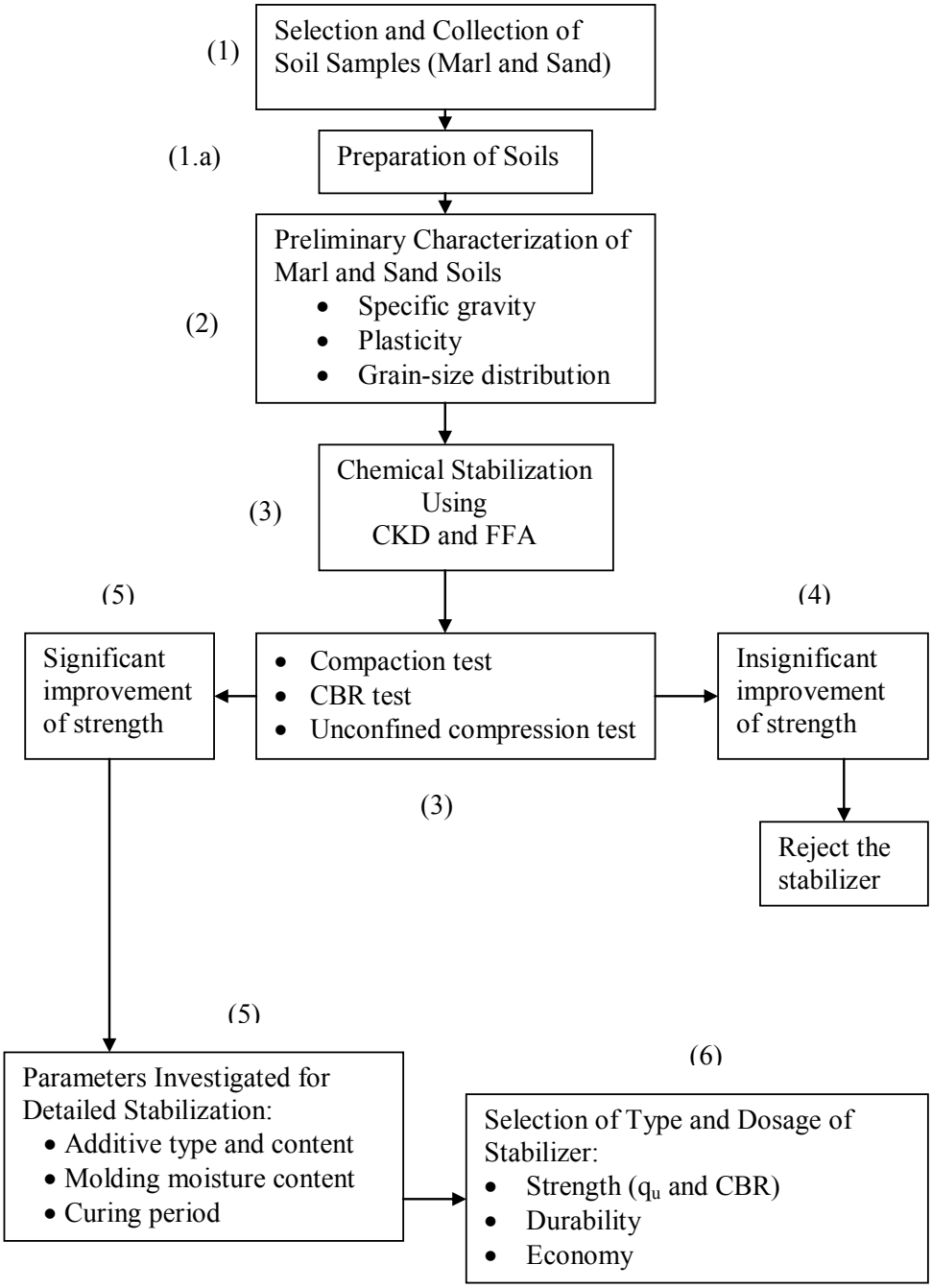

Figure 3-1: Flow Chart for the Experimental Program of this Stabilization Program

3.1.1 Preparation of Soil Samples

The soil samples were brought to the geotechnical laboratory and, thereafter, sieved through ASTM Sieve #4 and then oven dried. The soil materials were then thoroughly mixed and stored in plastic drums till testing.

3.2 Characterization of the Collected Samples

Preliminary characterization tests were performed to asses the basic engineering properties of the two collected soil samples. These preliminary tests included specific gravity, plasticity tests and grain size distribution. In addition, the compaction and strength characteristics were investigated by using modified Proctor compaction and California bearing ratio tests.

3.2.1 Specific Gravity Test

The specific gravity is needed for various calculation purposes in soil mechanics. It is used as a parameter in determination of some important properties of soil such as void ratio, unit weight of soil, and soil particle size analysis. Since both soil samples are sieved through ASTM Sieve #4, the test was performed in accordance with ASTM D 854. The test was conducted on two representative "disturbed" samples passing ASTM sieve #4 from each soil and the average of the samples was taken as the specific gravity value.

3.2.2 Plasticity Tests

The liquid limit and plastic limit tests were conducted on the material passing ASTM Sieve #40 using distilled water. The two tests were conducted in general accordance with ASTM D 423 and ASTM D 424, respectively. It was not possible to get the required number of blows for the liquid limit test for both soils, therefore, the liquid

limit was reported as "not defined". The two soils also could not be rolled to a thread of 1/8-in (3.18 mm), therefore, the soil was classified as "non-plastic".

3.2.3 Grain Size Distribution Test

This test is a basic requirement in any soil investigation. It is also essential in almost all soil classification systems. Both dry and washed sieving techniques (ASTM D 422) were used for the two types of soil.

In the wet sieving method, a representative soil sample was taken and washed through a set of sieves including ASTM No. 10, 20, 40, 60, 100 and 200 sieves until the water passing through each sieve was clear. The soil portion retained on each sieve as well as that passing through No. 200 sieve were dried in the oven and then weighed. The difference in weights of the (sieves + dry soils) and the (empty sieves) was used to determine the percentage passing for each sieve.

3.2.4 Compaction Test

The purpose of any compaction test is to determine the compaction characteristics especially the optimum moisture content at which the maximum dry density of the soil is attained. This test provides a relationship between dry density and moisture content for a given compaction method. Depending on the grain size distribution of the soil sample, different compaction testing procedures are used. In this investigation, the modified Proctor compaction test (ASTM D 1557) was used, as illustrated below.

The following procedure was followed when compacting the mixtures of soils (non-plastic marl or sand) with cement kiln dust or fuel fly ash and water: The required amount of soil was placed in Hobort mixer (0.3 m^3 capacity), the dosage of additive was added by weight of oven-dry (105° C) soil. Mixing was, thereafter, started in a dry state

for 1 minute, the water was then added to the mixture and mixing was continued for about another 3 minutes till the whole mixture was totally mixed and the final product was homogeneous. Compaction was made in five layers in the CBR mold. The CBR mold has a height of 5 in (127 mm) and a diameter of 6 in (152 mm) and the number of blows per layer was 56.

3.2.5 Unsoaked CBR Test

California bearing ratio (CBR) test was originally developed in California, USA, as a means to evaluate the suitability of a soil to be used as a subgrade material in pavements and, thereafter, adapted by the engineering communities as a test to empirically measure the strength of soil under controlled moisture and density conditions. The test is recognized worldwide because of its simplicity and applicability. Therefore, the test can easily be used to quantify the material for use in pavement construction.

In this investigation, CBR tests were conducted in compliance with ASTM D 1883. All samples prepared for moisture-density relationship was subjected to unsoaked CBR testing procedure. After sample preparation, the samples were sealed by plastic sheets and left to cure in laboratory conditions (23 ± 3°C) for 7 days and then tested. Figure 3.2 and Figure 3.3 show the machine used in the compaction and CBR tests and some of the sealed samples, respectively.

3.2.6 Unconfined Compression Test

Unconfined compressive strength (q_u) has commonly been used for the evaluation of the chemically-stabilized soils as well as untreated ones. q_u test is frequently used in many standards and codes for stabilized earth materials. Usually, a minimum q_u value is specified for different applications [Al-Amoudi, 2002]. q_u was adopted as a basic test in

this investigation where a comparative study of the effects of curing conditions and curing period on the strength gain of the stabilized soils has been studied.

In this investigation, specimens with h/d of 2 were prepared for all unconfined compressive testing. The required soil and additive content were mixed first and then the corresponding (optimum) moisture content was added and mixed thoroughly in the mechanical mixer for 3 minutes. Thereafter, the mix was compacted in a mold of 50 mm diameter and 100 mm height (h/d = 2) to the maximum dry density of the treated soil according to the modified Proctor compaction test. The mold used was of a split type with longitudinal slit along its axis. The slit was tightened and opened with the help of bolts. After compaction, the specimen was taken out of the mold by loosening the bolts. The specimens were wrapped in three layers of nylon sheets in order to inhibit any loss of moisture from the specimens. The samples were then put on the table in the laboratory and kept to cure for different curing periods (3, 7, 14 and 28 days) at the laboratory temperature (23 ± 3°C).

3.2.7 Durability Tests (Wetting and Drying)

Moisture, combined with temperature, can produce wet and dry or freeze or thaw cycles. The stabilized soils need to be strong and should maintain stability and durability to resist physical loads under the cyclic environmental loading and different exposure conditions. Rise and fall of water table, irrigation water, septic tanks, leakage from adjacent utilities, and seasonal variation of rainfall are responsible for these wetting and drying cycles. These conditions cause weight loss and/or volume change which in turn induce tensile and compressive stresses in the stabilized soils [Al-Ayedi, E. S. 1996].

Figure 3-2: Motorized Machine Used for CBR Test.

Figure 3-3: Some of CBR Sealed Specimens during the Curing Period.

In this investigation, the durability of stabilized soils was evaluated using two different procedures, the standard ASTM D 559 and the proposed slake durability test [Goodman, 1980]. The latter was originally used for rock testing but has recently been modified by Aiban et al. [1995] in order to accommodate the stabilized soil specimens with specific sizes.

3.2.7.1 Standard Durability Test (ASTM D 559)

Specimens of marl soil stabilized with CKD and FFA and sand stabilized only with CKD were prepared with different percentages as shown in Table 3.1. The mold used to prepare the soil samples was 4 in. (101.6 mm) in diameter and 4.6 in. (116.8 mm) in height. Each specimen was compacted in three layers to its modified Proctor maximum dry density. The number of blows was adjusted to get the same modified Proctor maximum dry density. After many trials, the number of blows was found to be 39 blows for each layer. After compaction, all samples were extruded from the molds. Four samples were prepared for each mix. Two of these samples were designated as the weight loss samples, while the other two were designated as the volume change samples. The height and diameter for the volume change samples were recorded.

All samples were cured for seven days at the laboratory temperature ($23 \pm 3^{o}C$) and 100% relative humidity. Thereafter, the samples were placed in a water tank for 5 hours at room temperature and, thereafter, transferred to an oven at $71^{o}C$ and kept there for 42 hours. This process constitutes one cycle of wetting and drying for the stabilized soils. At the end of this cycle, the specimens designated as volume change were dimensioned using a vernier caliper, and were weighed. The other two specimens were brushed using a standard brush with two strokes on the whole surface with a force of

about 3 Ib. (1.36 kg). To apply the 3 Ib. (1.36 kg) force, each sample was placed on a balance, and was then brushed while observing the specified force on the scale of the balance. The weight of the samples before and after brushing was measured. Similar measurements were taken for the remaining 11 cycles thus subjected each specimen to 12 cycles according to the standard ASTM D 559. At the end of each cycle, the weight loss and volume change for the respective specimens were noted. At the end of 12 cycles, the samples were dried to a constant weight at 110°C. Therefore, the volume change and weight loss were determined according to the following two equations (ASTM D 559):

i) Volume change (VC):

$$\text{VC (\%)} = \left(\frac{V_i - V_f}{V_i}\right) * 100 \qquad (3.1)$$

ii) Weight loss (WL):

$$\text{WL (\%)} = \left(\frac{W_i - W_f}{W_i}\right) * 100 \qquad (3.2)$$

Where:

VC = volume change of the specimen after f cycles (%);

V_i = initial volume of the specimen (cm^3);

V_f = final volume of the specimen (cm^3);

WL = weight loss of the specimen after f cycles (%);

W_i = initial calculated oven-dry weight (kg); and

W_f = final corrected oven-dry weight (kg).

Moreover, it is worth mentioning that a correction was applied on the oven-dry weight, which could be determined according to the following equation (ASTM D 559):

Corrected oven-dry weight = A/B * 100 (3.3)

Where:

A = oven-dry weight after drying at 230°F (110°C); and

B = percentage of water retained on specimen plus 100.

3.2.7.2 Slake Durability Test

This test is basically used to determine rocks durability [Goodman, 1980]. A certain weight (500 gm) of rock pieces is placed in a drum made of 2 mm stainless steel mesh. The drum is 140 mm in diameter and 100 mm in length. The drum is rotated at a speed 20 rpm, while being partially submerged in water. The weight loss after 10 min. of rotation is a measure of the durability of the rock.

In this investigation, this test was adopted and modified for the stabilized soil specimens [Aiban et al. 1995]. The diameter and the length of the drum were changed to 304.8 mm (12 inch) and 152.4 mm (6 inch), respectively. To allow the soil specimens to travel the same distance as by the rock piece in the original test, the number of revolutions was adjusted to accommodate the change in dimensions. The revolution time was reduced to 4.6 min. instead of 10 min.. This new arrangement would give a total travel distance of 88 m similar that of original test. The set-up of slake durability test is shown in Fig. 3.4.

Two additional samples were compacted for each percentage of additives. These samples were subjected to the same wet and dry cycles as for the samples tested using ASTM D 559 durability test but tested using the modified slake durability apparatus. After slaking, the surface of the sample was cleaned with a dry absorbent cloth and then weighed. The weight loss for each sample was determined by taking the weight before

and after the slaking in each cycle. After 12 cycles, the samples were oven dried at 110°C to get the volume change and weight loss according to the ASTM D 559 equations, as illustrated before.

Table 3-1: CKD and FFA Percentages Used in Durability Test.

Non-plastic Marl Soil				Sand Soil	
CKD Stabilization		FFA Stabilization		CKD Stabilization	
Cement (%)	CKD (%)	Cement (%)	FFA (%)	Cement (%)	CKD (%)
2	5	5	5	2	5
2	10	5	10	2	10
2	20	5	15	2	20
0	5	0	5	0	10
0	10	0	10	0	20
0	15	0	15	0	30
0	20	-	-	-	-
0	30	-	-	-	-

3.3 Stabilization of Non-plastic Marl and Sand Soils

Stabilization of non-plastic marl and sand using cement kiln dust (CKD) or fuel fly ash (FFA) is the main objective of this investigation. Chemical stabilization involves mixing the soil with one or a combination of chemical admixtures, for the general goal of improving or controlling its volume stability, strength and stress-strain behavior and durability [Winterkon and Pamukcu, 1992]. Admixtures can be in the form of liquid,

powder, or slurry. The most commonly used chemical admixtures are Portland cement, lime, fly ash, and bitumen.

In this investigation, CKD and FFA were used as chemical admixtures. These additives are considered as waste materials and it is a noble task to use them in soil stabilization. The fundamental processes that take place in chemically stabilized soil system are cementation and ion-exchange reactions, alteration of soil surface properties, plugging of voids, and coating the soil particles thereby binding them together [Winterkon and Pamukcu, 1992]. Chemical stabilization technique is considered to be relatively more economical and cheaper than many other techniques and requires less expertise and tools. Out of these two additives used in this research, the suitable chemical additive (i.e. the one producing high strength using the CBR test), would be chosen.

3.3.1 Optimization of Non-plastic Marl and Sand Stabilization

The main objective of chemical stabilization is to improve the engineering properties of the soils. The degree of improvement is different from project to project and from soil to soil. The improvement depends on the amount and type of a stabilizer, the environmental conditions, and the construction conditions as well as the properties of the soil itself. Considering all conditions which contribute positively or negatively, an optimum level of stabilizer should be determined which should also be economical and satisfy the minimum requirements of the strength and durability.

Marl and sand can be used as construction materials for base and sub-base of pavement. Strength, settlement, and durability are the main concerns in pavement structures. Strength of stabilized soil can be expressed in terms of unconfined compressive strength and CBR and can be improved using chemical additives. There are,

however, certain ranges of moisture and temperature for which these parameters can be maximized.

In this investigation, to maximize the strength of the stabilized soils, the following parameters were investigated:

1. Soil type (i.e. marl and sand).
2. Additive type and content (i.e. CKD and FFA).
3. Molding moisture content.
4. Curing period (i.e. 3, 7, 14, and 28 days).

The effects of these parameters on strength and durability were assessed using one or more of the following tests:

1. Compaction.
2. California bearing ratio (CBR).
3. Unconfined compressive strength (q_u).
4. Durability.

These parameters along with their beneficial effects on the optimization process are discussed in the following paragraphs.

3.3.2 Additives Content

In this investigation, additive content is defined as the percentage of the weight of additive (CKD or FFA) to that of oven-dry soil plus additive. Because additives are expensive materials, it is importance to determine an optimum value, which depends on the soil type and intended goal. In this research, CKD and FFA are considered as waste materials and apart from the economic point of view, the injudicious increase in additive content may lead to negative effects.

Figure 3-4: Set-up for Modified Slake Durability Testing of Soil-Cement Specimens (Aiban et al., 1999).

In this research, the different percentages of CKD and FFA additives used to determine the optimum content to be mixed with the "parent" non-plastic marl and sand (0% cement) and those treated with 2 and 5% Portland cement are shown in Table 3.2 and Table 3.3 for CKD and FFA, respectively. Each of these percentages was used in the moisture-density relationship test to determine the maximum dry density and optimum moisture content. The desired additive content was chosen based on the accepted results of unconfined compressive strength of stabilized specimens as well as on the durability requirements. Economy would be briefly addressed by comparing the addition of CKD and FFA (very cheap) with cement (very expensive) if and when the strength and durability requirements were fulfilled.

3.3.3 Curing Conditions

Chemical stabilization processes need water for hydration and producing cementitious materials, known as cementing gel. For cement and CKD stabilization, the hydration reaction is initially fast, especially during the first 7 days, but it decreases with time and depends on the curing conditions and ambient temperature.

In the field, in order to prevent moisture loss, different methods are used. These methods may include covering the surface of stabilized soils with a wet layer of sand or sealing by spraying curing compounds such as emulsified asphalt. Water is regularly sprinkled to supplement the moisture loss from the stabilized soil and enhance the hydration reactions.

In this investigation, the samples were wrapped with 3 layers of plastic sheet, to ensure that no moisture loss takes place. Thereafter, these wrapped samples were placed in the laboratory conditions till testing. Fig. 3.5 shows part of the wrapped samples.

3.3.4 Curing Period

Though the rate of hydration of cementitious materials decreases with time, the hydration process will continue indefinitely. This process is responsible for the strength gain with time for stabilized soils with additives that have cementitious materials such as cement and CKD. Usually, the 3-day unconfined compressive strength is used for quality control purposes during field construction while the 7-day strength is used as the main criterion for design purposes (Bahtia, 1967). Strength after 28 days of curing can also be used to asses the bearing capacity of an existing pavement.

In this program, four different curing periods, namely 3, 7, 14, and 28 days, were used and the effect of these periods on the strength of stabilized soils was studied

Table 3-2: CKD Percentages Used in Non-plastic Marl and Sand Stabilization.

Cement (%)	CKD (%)
2	0
2	5
2	10
2	20
0	5
0	10
0	15
0	20
0	30

Table 3-3: FFA Percentages Used in Non-plastic Marl and Sand Stabilization.

Cement (%)	FFA (%)
5	0
5	5
5	10
5	15
0	5
0	10
0	15

Figure 3-5: Some of Wrapped Stabilized Specimens Used in q_u Test.

Chapter 4

Results and Discussion

In this chapter, the results of the experimental work are presented and appropriate interpretations for such data are addressed to explain the reasoning for the behavior of the soils, non-plastic marl and sand, before and after stabilization using CKD and FFA additives.

4.1 Characterization of Marl Soil

Characterization tests were done according to the relevant ASTM and AASHTO standards in order to identify the selected eastern Saudi marl with respect to its particle size, its plasticity and its suitability as a bearing soil for roads, highway and foundations. Characterization tests in this investigation program included specific gravity of the solid grains, grain size distribution and plasticity tests.

4.1.1 Specific Gravity Test Results

Two specific gravity tests were conducted and the values obtained are 2.7 and 2.68 with an average value of 2.69 ≈ 2.70 for marl soil. Since the variation from the average was minimal, the results were consistent. The value of the specific gravity falls within the rang of eastern Saudi Arabia marl as reported by Ahmed (1995).

4.1.2 Plasticity Tests

Liquid and plastic limits were conducted according to the ASTM D 423 and ASTM D 424, respectively. For this kind of marl soil, it was not possible to get the number of blows for the liquid limit test, so it was reported as nil. The soil also could not

be rolled to a thread of 1/8 in (3.18 mm), therefore, the soil was classified as non-plastic marl.

4.1.3 Grain-Size Distribution Test Results and Classification

The grain-size distribution curves are presented in Fig.4.1. It can be seen that the percent passing Sieve No. 200 is 10.6 and 29 when the marl samples were sieved dry and wet methods, respectively. The sediments in the marl soil were non-plastic; therefore, the soil could be classified as SM according to the USCS system. However, according to the AASHTO soil classification system, the soil could be classified as A-3 based on both dry and washed sieving.

Figure 4.1 indicates that the grain-size curve obtained when using wet sieving method was consistently above the one when dry sieving method was used. This is attributed to the fact that water tends to dissolve the bonds and salts between particles of the soil, thus, the percent passing of the soil is higher than that for dry sieving.

4.2 Chemical Analysis of Additives

Ordinary Portland cement (OPC) type one, fuel fly ash (FFA) from Shuaibah Power Plant and cement kiln dust (CKD) from Arabian Cement Company Ltd. (ACCL) additives were used in this investigation. The chemical analyses of these additives are shown in Table 4.1 and Table 4.2 below.

4.3 Chemical Stabilization Test Results of Non-Plastic Marl Soil

Chemical stabilization involves of mixing the parent soil with one or a combination of chemical admixtures, for the general goal of improving or controlling its volume stability, strength, stress-strain behavior and durability [Winterkon and Pamukcu,

1992]. Admixtures can be in the form of liquid, powder or slurry. The most commonly used chemical admixtures are Portland cement, lime, fly ash, and bitumen.

In this research, CKD and FFA were used as chemical admixtures. These additives are considered as waste materials and it is a noble task to use them in soil stabilization. Chemical stabilization technique is considered to be relatively more economical and cheaper than many other techniques and require less expertise and equipment.

Table 4-1: Elemental Composition of OPC and FFA.

Element	**OPC**		**FFA**	
	Weight (%)	**Atomic (%)**	**Weight (%)**	**Atomic (%)**
Oxygen (O)	46.18	66.27	29.68	31.66
Carbon (C)	Nil	Nil	32.52	46.20
Magnesium (Mg)	3.11	2.94	19.20	13.48
Aluminium (Al)	0.71	0.60	0.44	0.28
Silicon (Si)	6.94	5.68	0.33	0.20
Sulphur (S)	1.39	1.00	11.42	6.08
Calcium (Ca)	39.45	22.60	0.31	0.13
Iron (Fe)	2.22	0.91	0.50	0.15
Vanadium (V)	Nil	Nil	4.11	1.38
Chromium (Cr)	Nil	Nil	0.08	0.03
Manganese (Mn)	Nil	Nil	0.41	0.13
Nickel (Ni)	Nil	Nil	1.01	0.29

Table 4-2: Chemical Analysis of ACCL-CKD

Constituent	Weight, %
Major	
CaO	49.3
SiO_2	17.1
Chloride	6.90
Loss on ignition	15.8
Minor	
Al_2O_3	4.24
Fe_2O_3	2.89
K_2O	2.18
MgO	1.14
Na_2O	3.84
P_2O_5	0.12
Equivalent alkalis (Na_2O+0.658K_2O)	5.27
SO_3	3.56
BaO (µg/g (ppm))	78.2
Cr_2O_3	0.011
CuO	0.029
NiO	0.012
SrO	0.37
TiO_2	0.34
V_2O_5	0.013
ZnO (µg/g (ppm))	65.8
ZrO_2	0.011

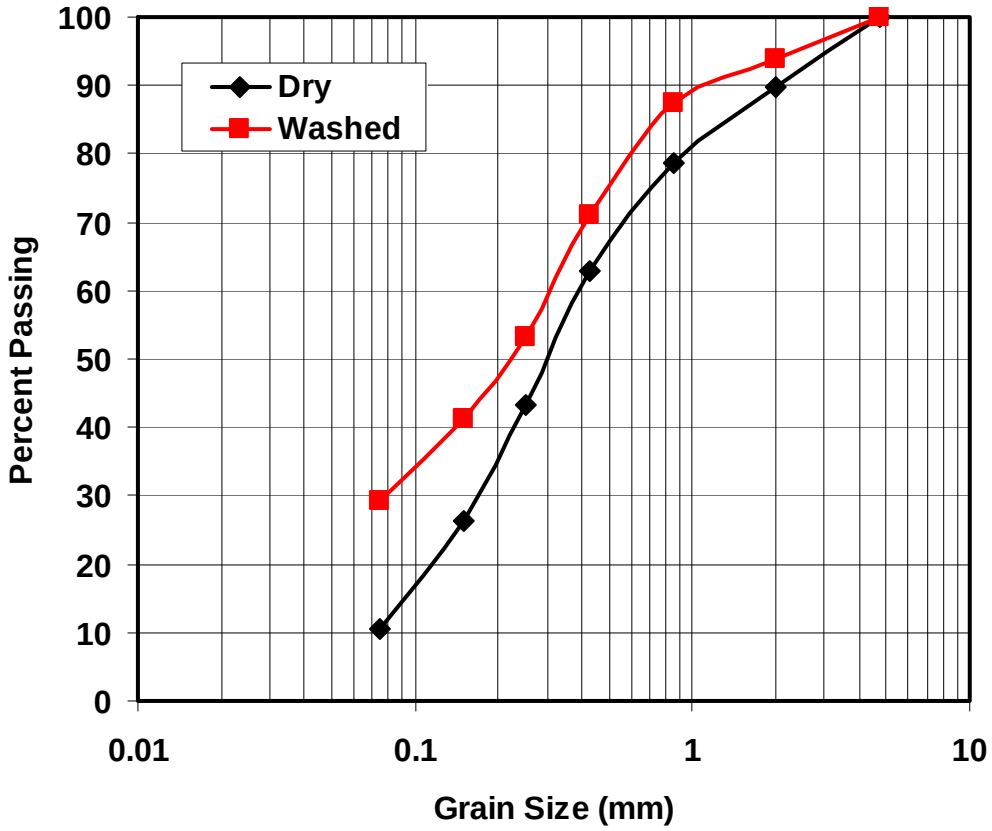

Figure 4-1: Grain-Size Distribution of Marl Soil

Out of these two additives used in this research, the suitable chemical additive (i.e. the one producing high strength using the CBR test), would be chosen.

4.3.1 Compaction Test Results of Non-plastic Marl

The moisture-density relationship reflects the behavior of soils during compaction. Dry density and moisture content control the structure of the soil which directly relates to the properties of the soil such as strength, compressibility, and permeability. Thus, the soil to be used as a construction material needs to be compacted to a certain dry density and moisture content.

In this research, compaction tests were conducted on plain (0% additive) soil as well as on non-plastic marl-CKD and non-plastic marl-FFA with additive contents in the range of 5 to 30%. Moreover, 2% and 5% cement additions were used as references to compare the CKD and FFA performance with. The results, presented in typical plots of water content versus dry unit weight (γ_d), are plotted in Figures 4.2, 4.3, 4.4, and 4.5 for CKD and FFA additions, respectively. A study of these figures reveals that for non-plastic marl it was expected to have an increase in the maximum dry unit weight due to the addition of cement since the specific gravity of cement is higher than marl soil, however, the situation here is different. Figures clearly show that cement addition led to a marginal decrease in the maximum dry unit weight. This is probably due to the formation of macropores which tend to reduce the density (but not the strength, as will be addressed later). From the same figures it can be noted that there is an increase in maximum dry unit weight of the non-plastic marl with the increase in CKD till 20%. Thereafter, there is a decrease in the maximum dry unit weight when the CKD content exceeds 20%. This is attributed to the fact that the CKD additive is very fine and it tends to fill up the voids

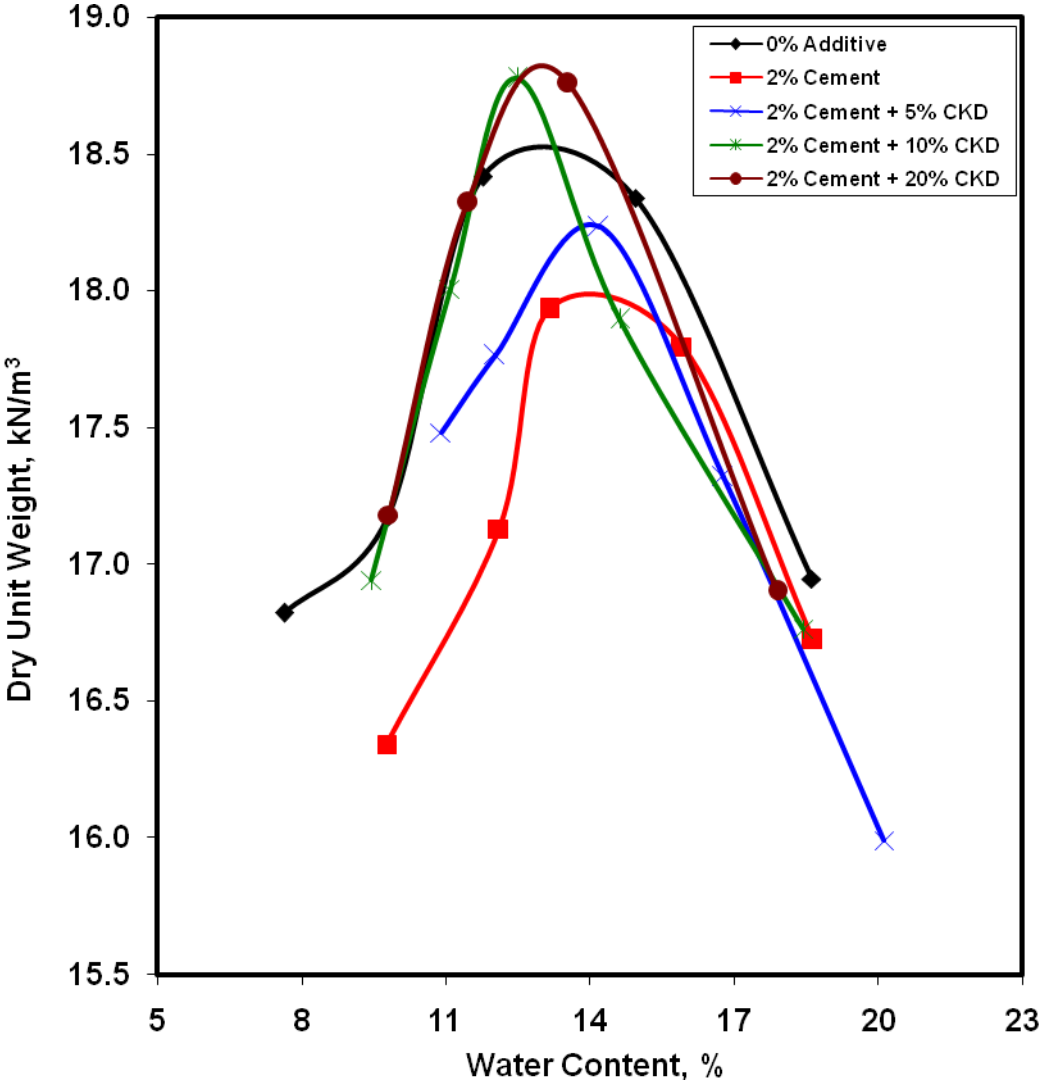

Figure 4-2: Effect of CKD Addition with 2% of Cement on Moisture-Unit Weight Relationship for Non-Plastic Marl

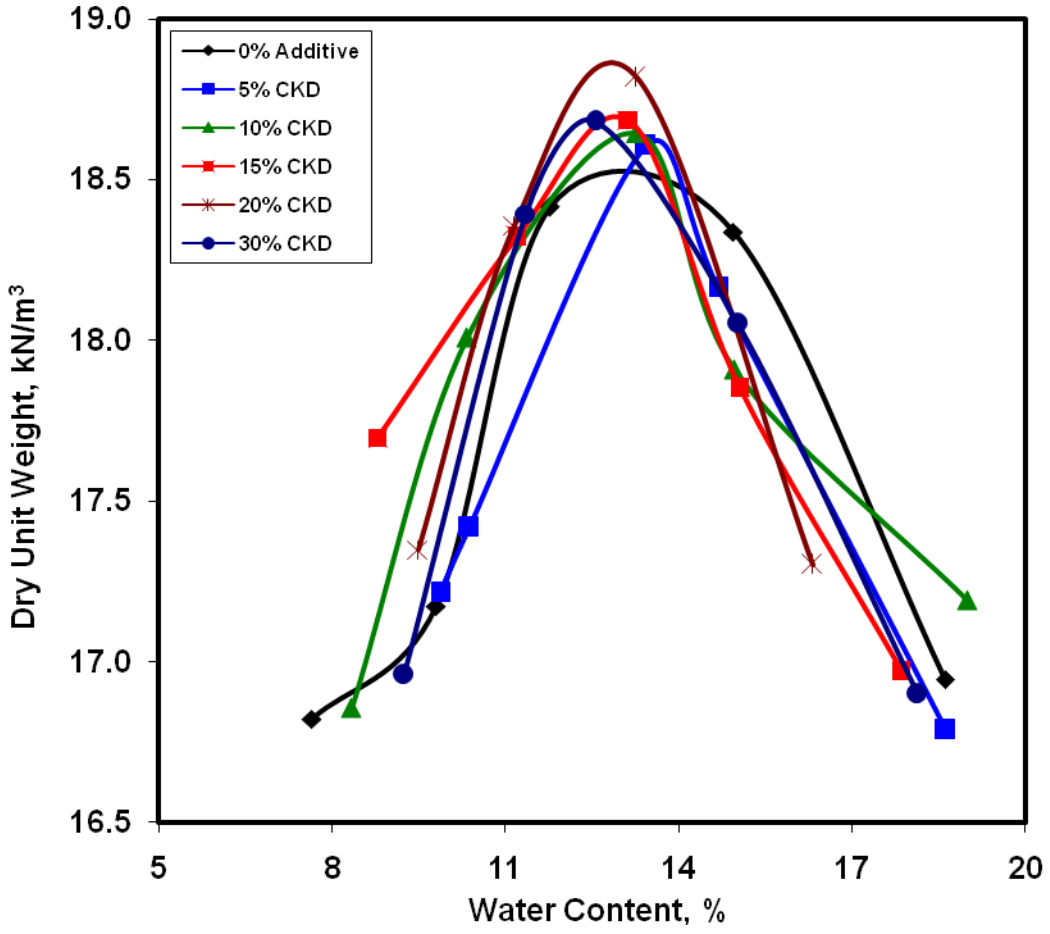

Figure 4-3: Effect of CKD Addition on Moisture-Unit Weight Relationship for Non-Plastic Marl

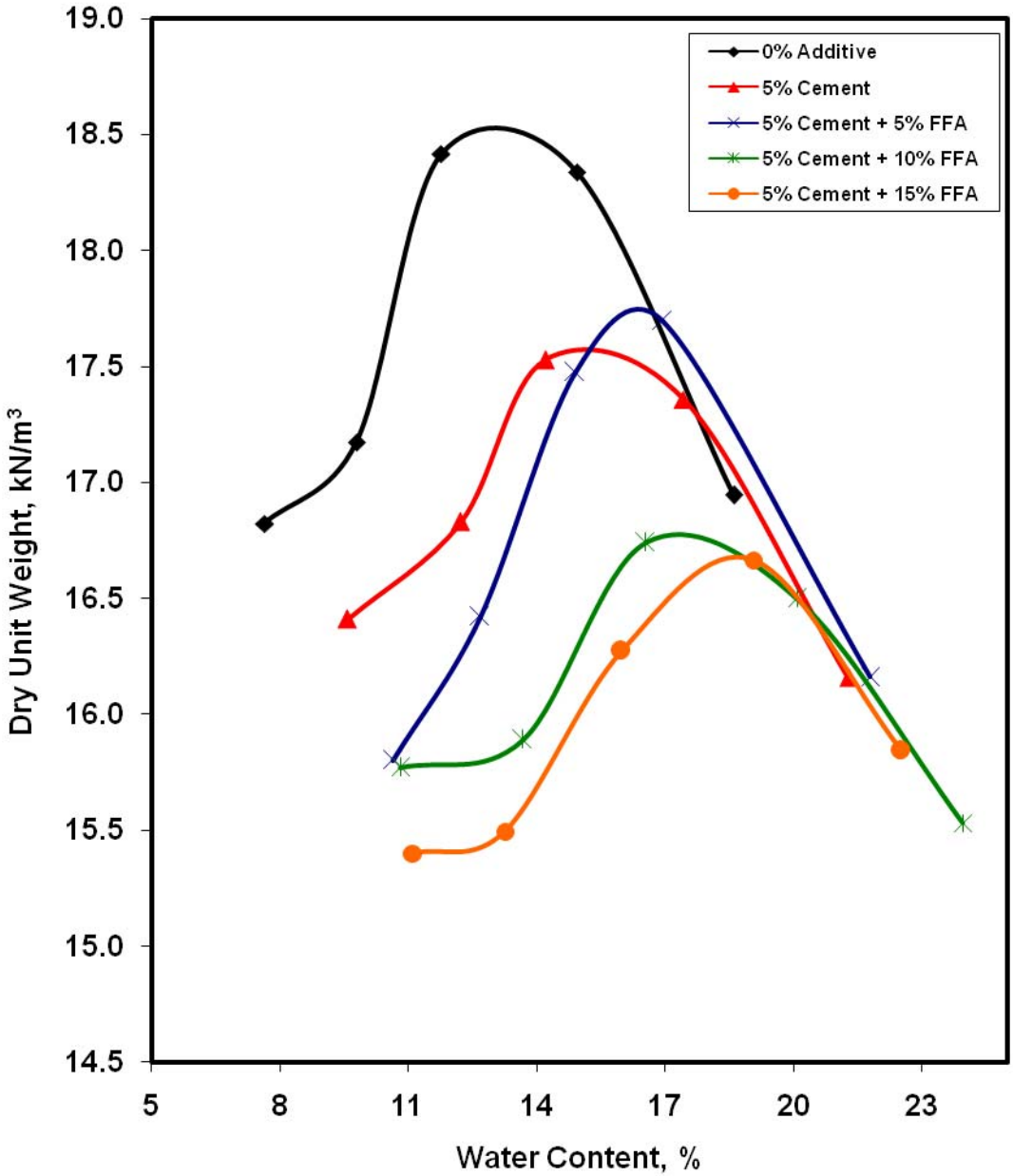

Figure 4-4: Effect of FFA Addition with 5% of Cement on Moisture-Unit Weight Relationship for Non-Plastic Marl

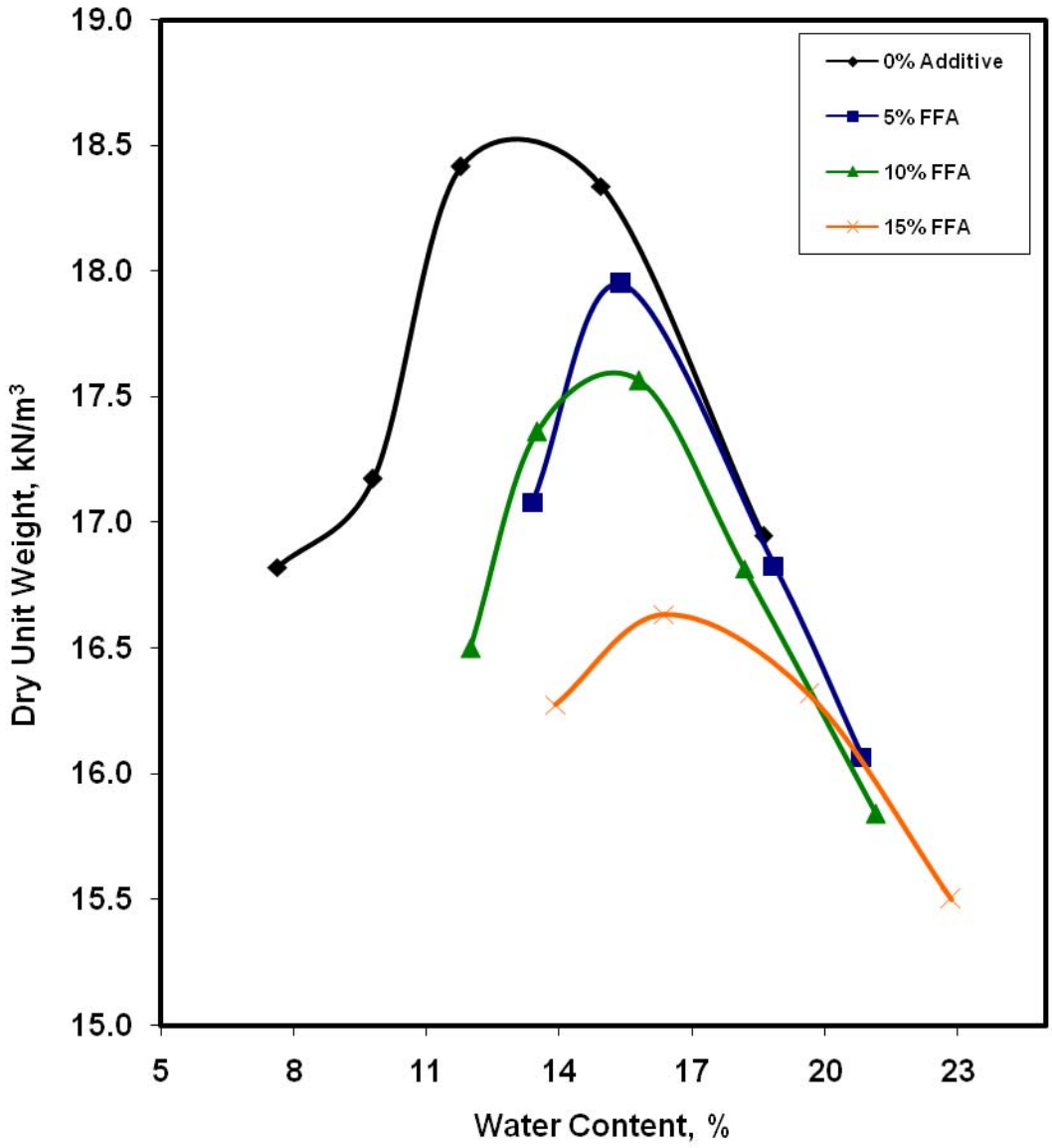

Figure 4-5: Effect of FFA Addition on Moisture-Unit Weight Relationship for Non-Plastic Marl

between the non-plastic marl particles and, thus, the dry unit weight increases. Further increases in CKD content disrupt the granular structure of the non-plastic marl, causing the particles to float in the CKD and thus reduce the dry unit weight. These figures also reveal that for non-plastic marl, there is a decreasing trend in the optimum moisture content of the non-plastic marl with the increase in CKD content. The decrease in optimum moisture content is more pronounced for higher percentages of the stabilizer. However, in the case of fuel fly ash (FFA) addition, the trend is reversed, whereby an increase in the FFA content resulted in a decrease in the maximum dry unit weight and an increase in the optimum moisture content. The increase in the optimum moisture content may be attributed to the fact that FFA is a very fine material, thus, any FFA addition needs more water for lubrication.

4.3.2 CBR Test Results

CBR tests are generally used to evaluate the suitability of a soil to be used as a subgrade material in pavements. Figure 4.6 shows the moisture-unit weight-CBR relationship for untreated non-plastic marl soil (0 additives). It is seen that the maximum dry unit weight $\{\gamma_{d(max)}\}$ was 18.5 kN/m^3 at an optimum moisture content of 13%. From the same figure, we can notice that the compaction curve follows the typical γ_d-w relationships whereby γ_d increased initially with the increase in moisture content until it reaches the maximum dry unit weight $\{\gamma_{d(max)}\}$ at the optimum moisture content (w_{opt}). Further increase in the moisture content resulted in a reduction in the dry unit weight. On the other hand, the maximum CBR was 47 at a moisture content of 11.8%. Similarly, we can observe from the data in Figure 4.6 that the CBR values increased with increasing the moisture content until the maximum CBR was attained at a moisture content of 11.8%

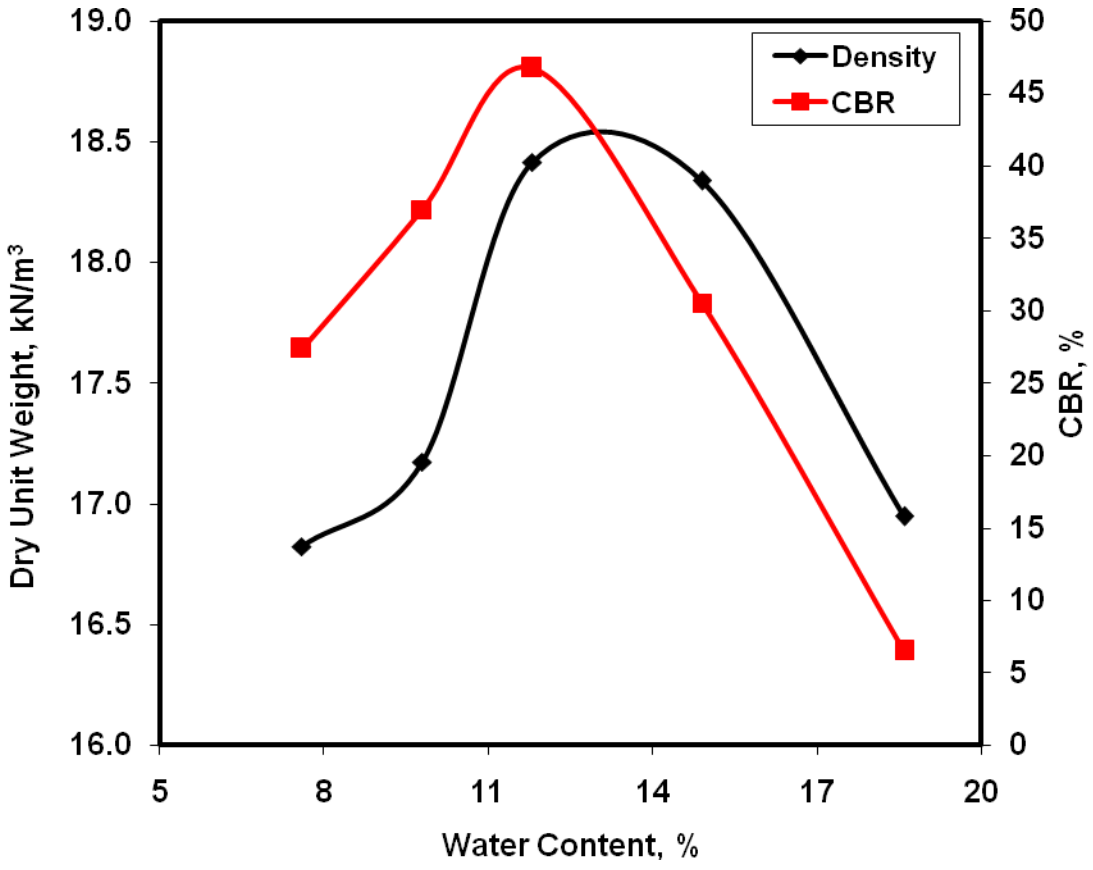

Figure 4-6: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil (0% Addition)

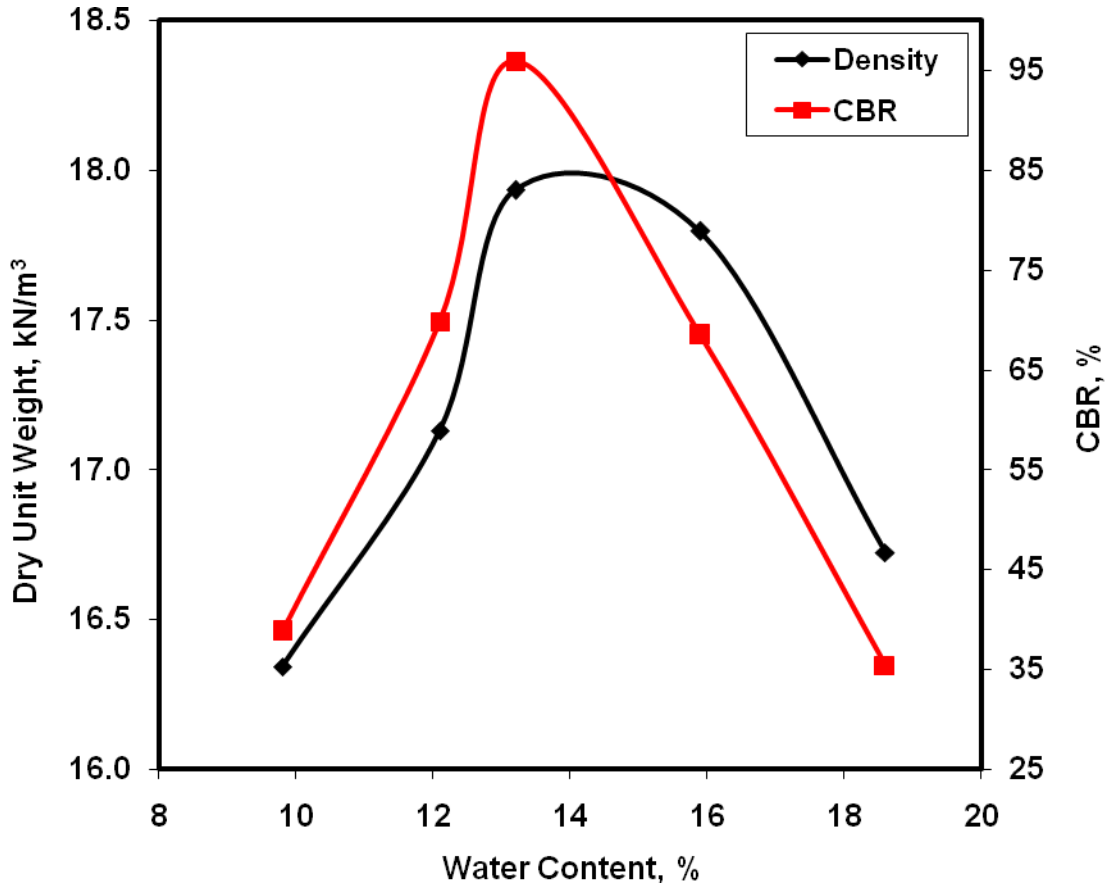

Figure 4-7: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 2% Cement Addition

After this value, the increase in moisture content led to a substantial reduction in the CBR value. The results indicated that the moisture content for maximum CBR is less than the optimum moisture content obtained from the dry unit weight-moisture content relationship (11.8% compared with 13%). This is in agreement with the findings that have been reported in the literature for [Al-Amoudi et al., 1992a and Aiban et al., 1995].

Figure 4.7 presents the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 2% cement. It is seen that the maximum dry unit weight was 18 kN/m^3 at an optimum moisture content of 14%. However, the maximum value of the CBR was 96 at a moisture content of 13.2%. Comparing the data of Figure 4.7 with that in Figure 4.6 indicates that there is a marginal decrease in the maximum dry unit weight and a significant increase in the CBR value due to the addition of 2% cement to the non-plastic marl. This is attributed to the fact that cement has self cementing characteristics and reacts with soil and developed a high strength due to the significant amounts of calcium hydroxide and secondary calcium silicate hydrate (C-S-H) in the hydration products [Aiban et al., 1995].

Furthermore, for the CBR curve, with the addition of 2% cement, the CBR values increased initially with increasing the moisture content until it reached the maximum CBR value. After that, further increase in the moisture content resulted in a sharp reduction in the CBR value reaching 35 at a moisture content of about 18.6%. It is observed that, despite the sharp reduction in CBR value at 18.6%, it is still much higher than that of non-treated non-plastic marl which was 6.5.

Figure 4.8 depicts the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 2% cement and 5% CKD. The results show that the maximum dry unit

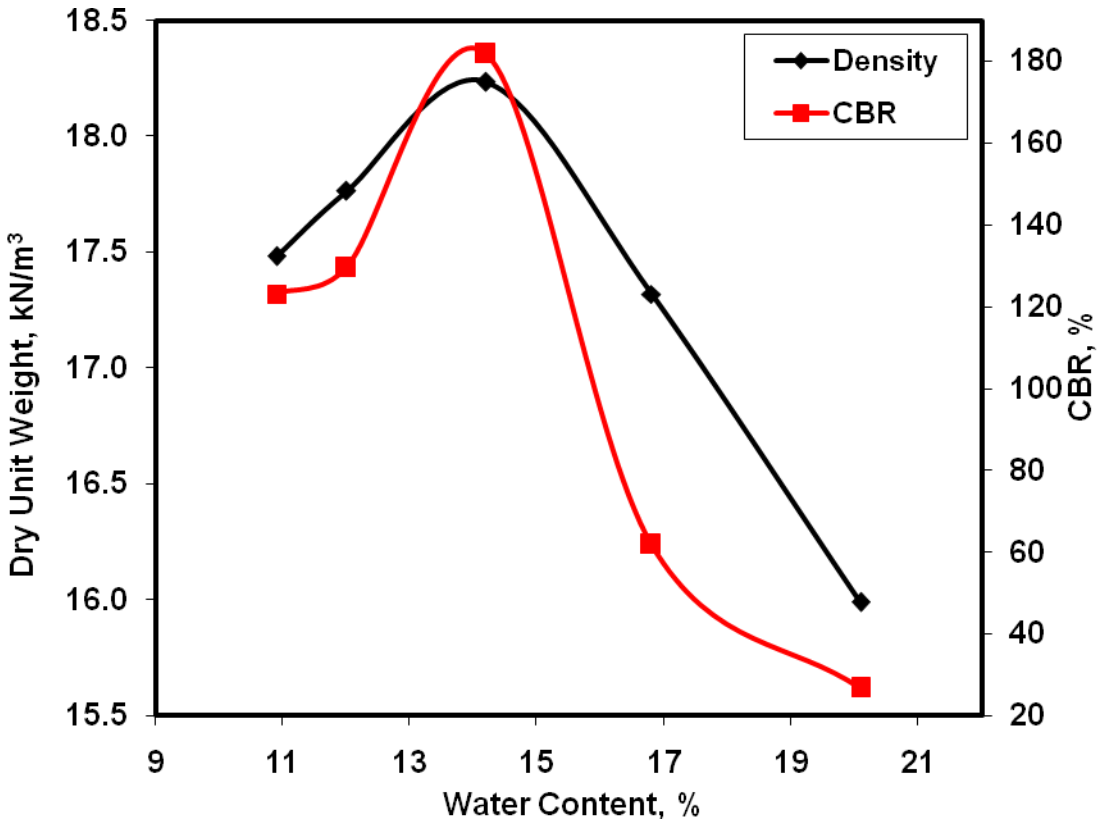

Figure 4-8: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 2% Cement and 5% CKD Additions

weight {$\gamma_{d(max)}$} was 18.2 kN/m^3 at an optimum moisture content of 14.2%. On the other hand, for the CBR- moisture curve, the maximum CBR value was 182 at a moisture content of about 14.2%. It is seen that the optimum moisture content is the same for maximum dry unit weight and CBR value. Comparison of the data of Figure 4.8 with that in Figure 4.7 indicates that there was a slight increase in the maximum dry unit weight and a significant increase in the CBR value. Further, the data indicates that there is a similarity between the CBR-moisture curve and dry unit weight-moisture curve. Moreover, for the CBR curve, with 2% cement and 5% CKD addition, the CBR at a water content of about 18.6% was about 80 compared with only 6.5 and 35.4 at the same moisture content when 0% and 2% cement additives were added, respectively.

Figure 4.9 shows the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 2% cement and 10% CKD. The results therein indicate that the maximum dry unit weight {$\gamma_{d(max)}$}was 18.8 kN/m^3 at an optimum moisture content of 12.5%. On the other hand, the maximum CBR value was 249 at a moisture content of 12.5%. There was an increase in both the maximum dry unit weight and maximum CBR value when the 2% cement and 10% CKD were added. Again, there is a similarity between the CBR-moisture curve and maximum dry unit weight-moisture curve. It is noticed that the $\gamma_{d(max)}$ as well as the maximum CBR value were attained at the same moisture content of about 12.5%.

Figure 4.10 presents the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 2% cement and 20% CKD. It is seen that the maximum dry unit weight was 18.8 kN/m^3 at an optimum moisture content of 13% while the maximum CBR value was 264. The maximum dry unit weight and maximum CBR value were

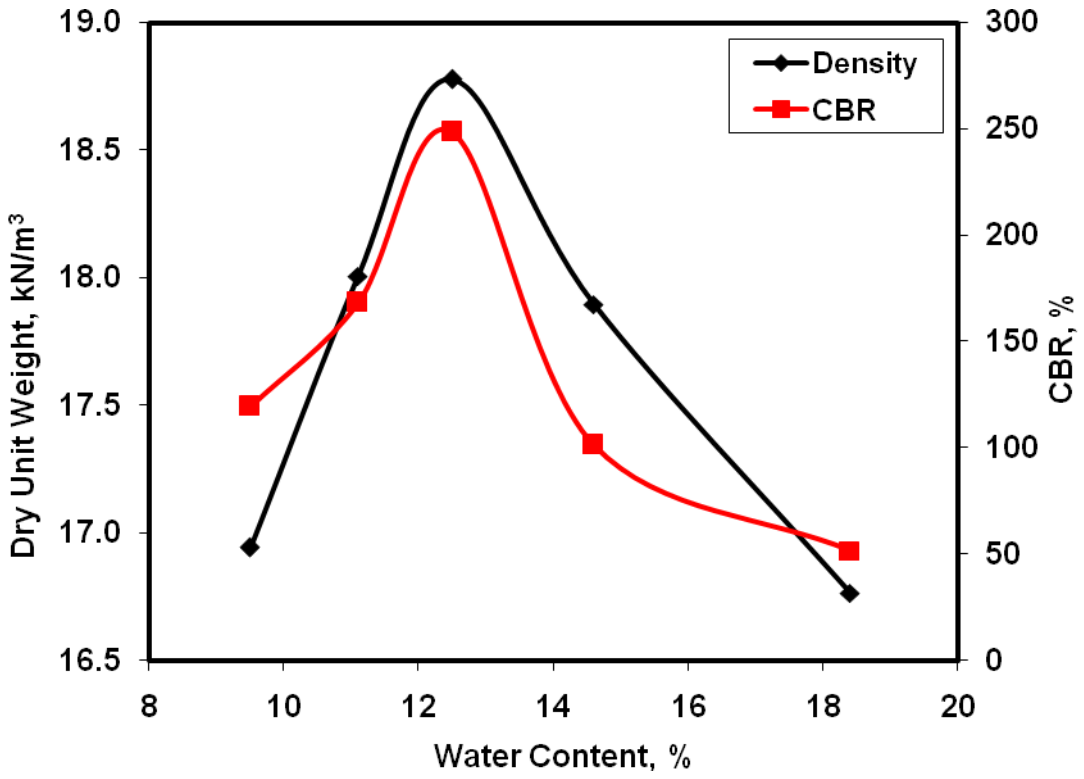

Figure 4-9: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 2% Cement and 10% CKD Additions

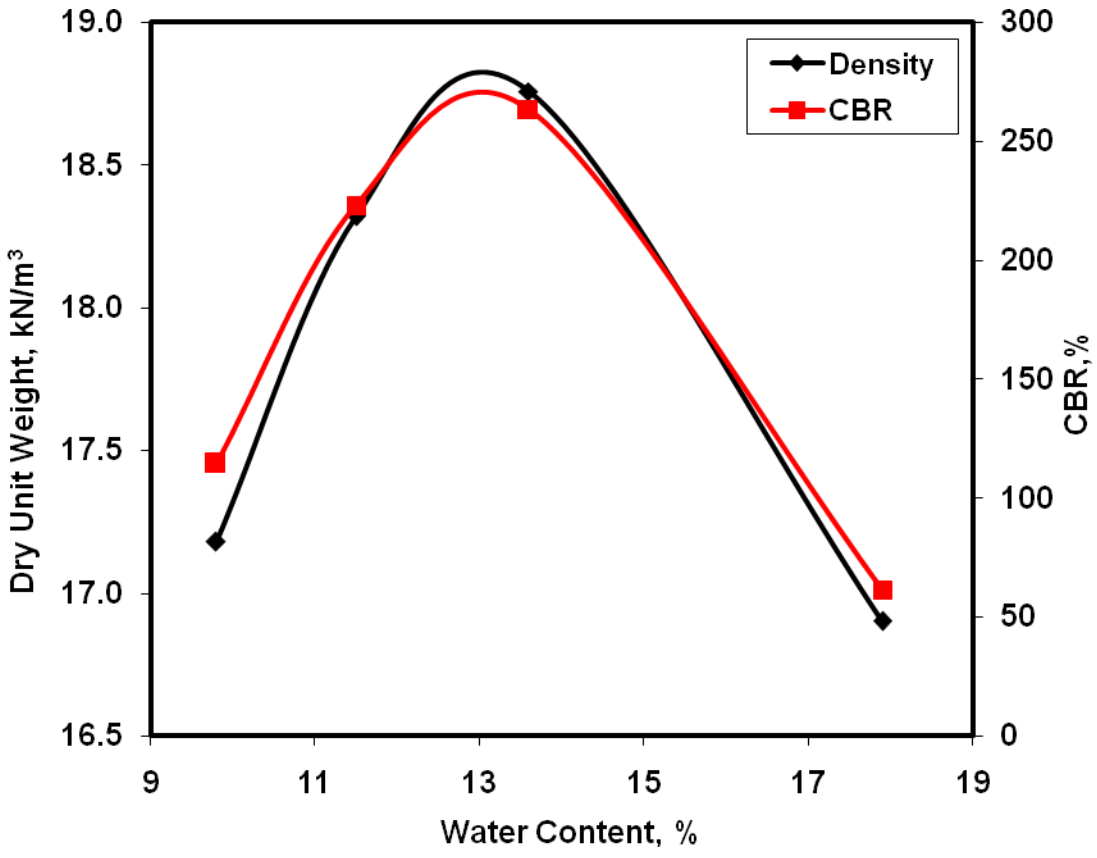

Figure 4-10: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 2% Cement and 20% CKD Additions

attained at an optimum moisture content of 13%. Comparison of the data of both curves in this figure indicates that there is an increase in the maximum dry unit weight as well as the maximum CBR value when 2% cement and 20% CKD additions were added.

Figure 4.11 depicts the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 5% CKD only (0% cement). The results indicate that there is no visible increase in the maximum dry unit weight compared with that of the plain non-plastic marl soil. The maximum dry unit weight was 18.6 kN/m^3 at an optimum moisture content of 13.4%. The results also indicate that there is an increase in the maximum CBR value, when 5% CKD was added, from 47 for 0% addition to 159. Furthermore, the ultimate value of the CBR at a moisture content of 18.6% increased significantly to 21.5 compared with 6.5 when the soil had no additives. It can also be seen that the maximum dry unit weight and maximum CBR value were attained at the same moisture content of 13.4%.

Figure 4.12 presents the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 10% CKD. It can be seen that the maximum dry unit weight was 18.6 kN/m^3 at an optimum moisture content of 13.2% while the maximum CBR value was 178 at a moisture content of 10.3%. Similarity is shown between the CBR-moisture curve and dry unit weight-moisture curve. However, the moisture content from the former curve is less than that from the latter one.

Figure 4.13 depicts the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 15% CKD. It can be seen from the Figure that the maximum dry unit weight was 18.7 kN/m^3 at an optimum moisture content of 13.1%. On the other hand, the maximum CBR value was 196 at a moisture content of 12%. The CBR-

moisture curve has the same trend of the dry unit weight-moisture content curve. The CBR value increased initially with increasing the moisture content until reaching the maximum value. Further increase of the moisture content led to a sharp reduction in the CBR values. This could be attributed to the sensitivity of the marl soil to water and this is in consensus with the finding that has been reported by Aiban et al. (1995). Comparison of the data in Figure 4.13 with that in Figure 4.12 indicates that there was an increase in the maximum CBR value with the increase in CKD addition from 10% to 15%, while the increase in the dry maximum dry unit weight was very marginal. On the other hand, the optimum moisture content seems to be less affected by the CKD addition.

Figure 4.14 depicts the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 20% CKD. The results indicate that the maximum dry unit weight was 18.8 kN/m^3 at an optimum moisture content of 13.3% while the maximum CBR value was 245 at almost the same moisture content. Comparison of the data in Figure 4.14 with that in Figure 4.13 indicates that there was a significant increase in CBR value from 196 for 15% CKD addition to 245 for the 20% CKD addition while the increase in the maximum dry unit weight was marginal. However, the increase in optimum moisture content is very marginal (13.1% compared with 13.3%).

Figure 4.15 shows the moisture-unit weight-CBR relationship for the stabilized non-plastic marl with 30% CKD. From the data in this figure, it is seen that the maximum dry unit weight was 18.7 kN/m^3 at an optimum moisture content of 12.6%. On the other hand, the maximum CBR value was 181 at a moisture content of 11.3%. The CBR-moisture curve follows the same trend of the dry unit weight-moisture curve. Initially the CBR value increased significantly with increasing the moisture content until reaching the

maximum value. Further increase in moisture content led to a sharp reduction in the CBR value. Comparison Figure 4.15 and Figure 4.14 indicates that there was a reduction in the maximum CBR value when the 30% CKD was added, however, the increase in maximum dry unit weight was negligible. On the other hand, the optimum moisture content decreased from 13.3% to 12.6% when the CKD addition increased from 20% to 30%, respectively. Again, the change in the optimum moisture content is marginal.

Figure 4.16, Figure 4.17 and Table 4.3 summarize the CBR test results for non-plastic marl stabilized with 2% cement together with various percentages of cement kiln dust and with various percentages of CKD alone, respectively. It is seen that the CBR value increased from 47 for 0% addition to 96, 182, 249, and 264 when 5, 10, and 20% of CKD were added with 2% cement by weight of dry soil, respectively, to non-plastic marl soil. Such obvious increase in the CBR values corresponds to an improvement ratio of 2.1, 3.9, 5.3, and 5.6 times that of the parent "plain" soil. Similarly, the CBR value increased from 47 for 0% addition of CKD (untreated soil) to 159, 178, 195, and 244 when 5, 10, 15, 20% of CKD by weight of dry soil, respectively, were added. This increasing in the CBR values corresponds to an improvement ratio of 3.4, 3.8, 4.2, and 5.2 times that of plain marl. Thereafter, the CBR value decreased from 244 for 20% CKD to 181 when 30% CKD was added to the non-plastic marl soil. However, the CBR value (181) for 30% CKD addition is still much higher than that of plain marl which was 47. The reduction in the maximum CBR value is most probably due to the further increases of CKD addition which in turn disrupt the granular structure of the non-plastic marl and cause the particles to float in the CKD and thus reduce the strength.

The results indicate that the moisture content for stabilized non-plastic marl with either CKD with 2% cement or CKD alone was higher than that for the untreated non-plastic marl (0% addition). This higher water content is attributed to the fact that the hydration kinetics of cement-CKD-non-plastic marl system requires an amount of water that is necessary for the proper compaction as well as to provide extra water for the hydration during the curing period [Al-Amoudi et al., 1995b].

It is well known that CKD has the self-cementing characteristics and reacts with soil in a manner similar to Portland cement. Furthermore, the hydration of the CKD produces significant amounts of calcium hydroxide and calcium silicate hydrate C-S-H. In addition, the fineness of CKD influences the strength development, especially at early ages [Millar and Azad, 2000]. From the previous discussion, one can conclude that the CKD addition enhanced the strength development in the marl soil significantly.

Figure 4.18 presents the moisture content-unit weight-CBR relationship for the stabilized non-plastic marl soil with 5% cement. The data reveals that the maximum dry unit weight was 17.5 kN/m^3 at an optimum moisture content of 14.2% while the maximum CBR value was 119 at the same moisture content. Comparison of the data in Figure 4.18 with that in Figure 4.6 and Figure 4.7 for untreated non-plastic marl and treated with 2% cement, respectively, indicates that there is an increase in the CBR value corresponding to an improvement ratio of 2.5 and 1.3 times that of untreated non-plastic marl and 2% cement addition, respectively.

The relationship between the maximum CBR values and CKD content for the non-plastic marl soil is depicted in Figure 4.19. It is noted that as the CKD content increases the maximum CBR value increases till 20% of CKD. Thereafter, the maximum

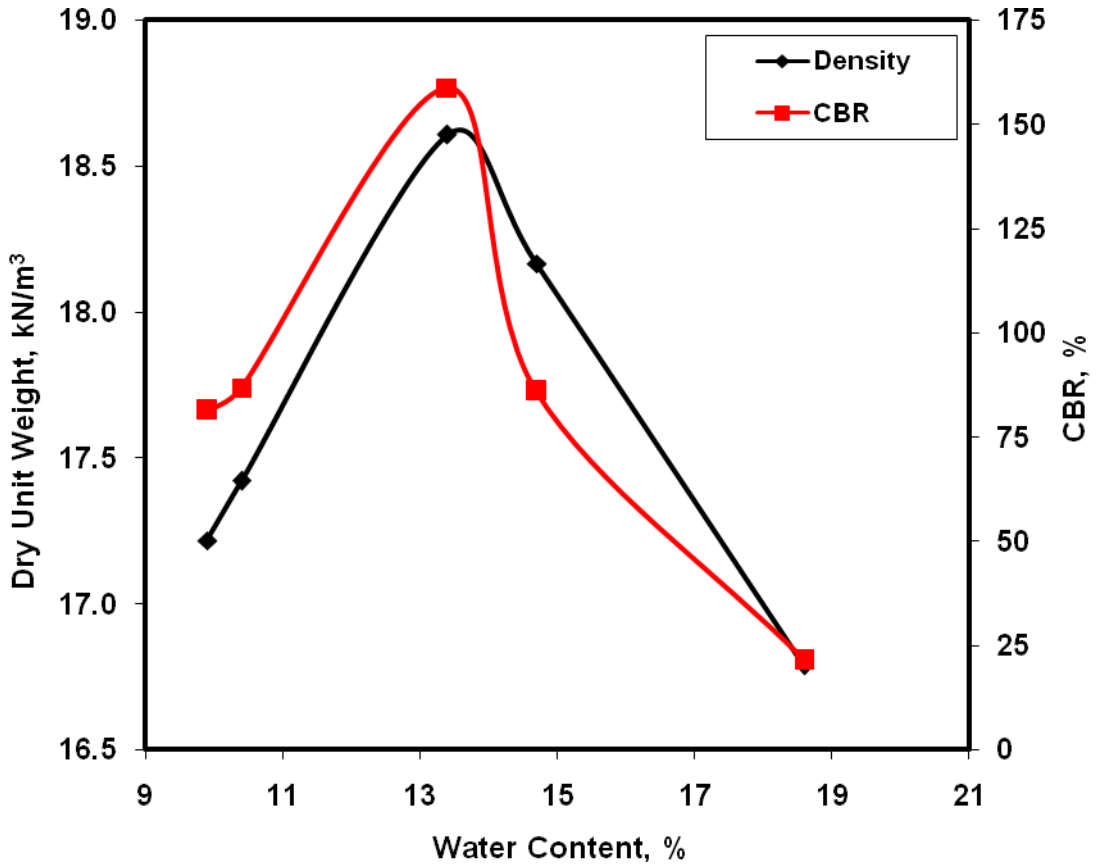

Figure 4-11: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% CKD Addition

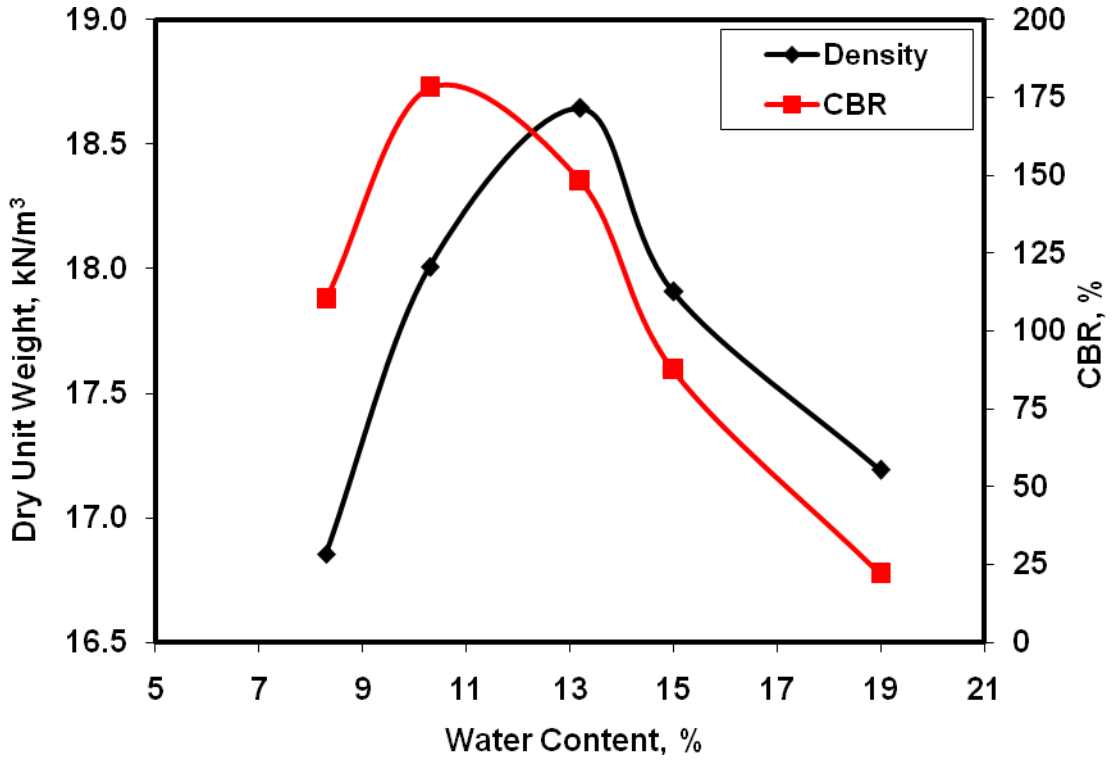

Figure 4-12: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 10% CKD Addition

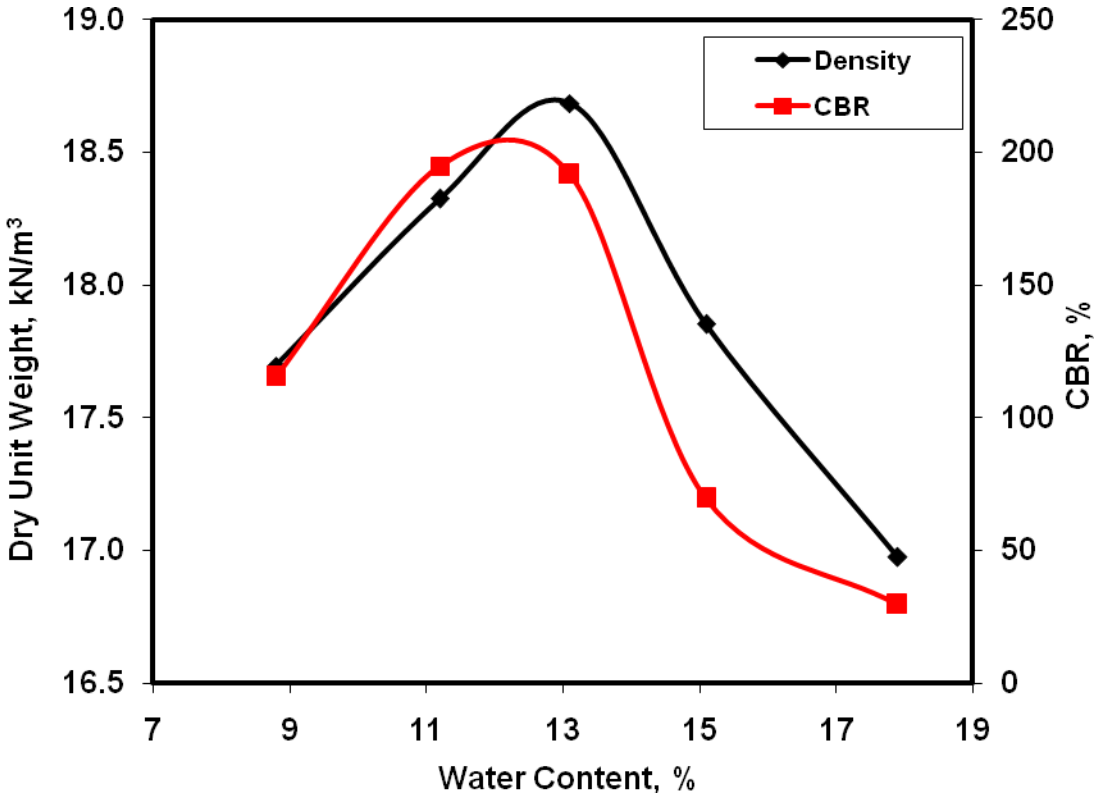

Figure 4-13: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 15% CKD Addition

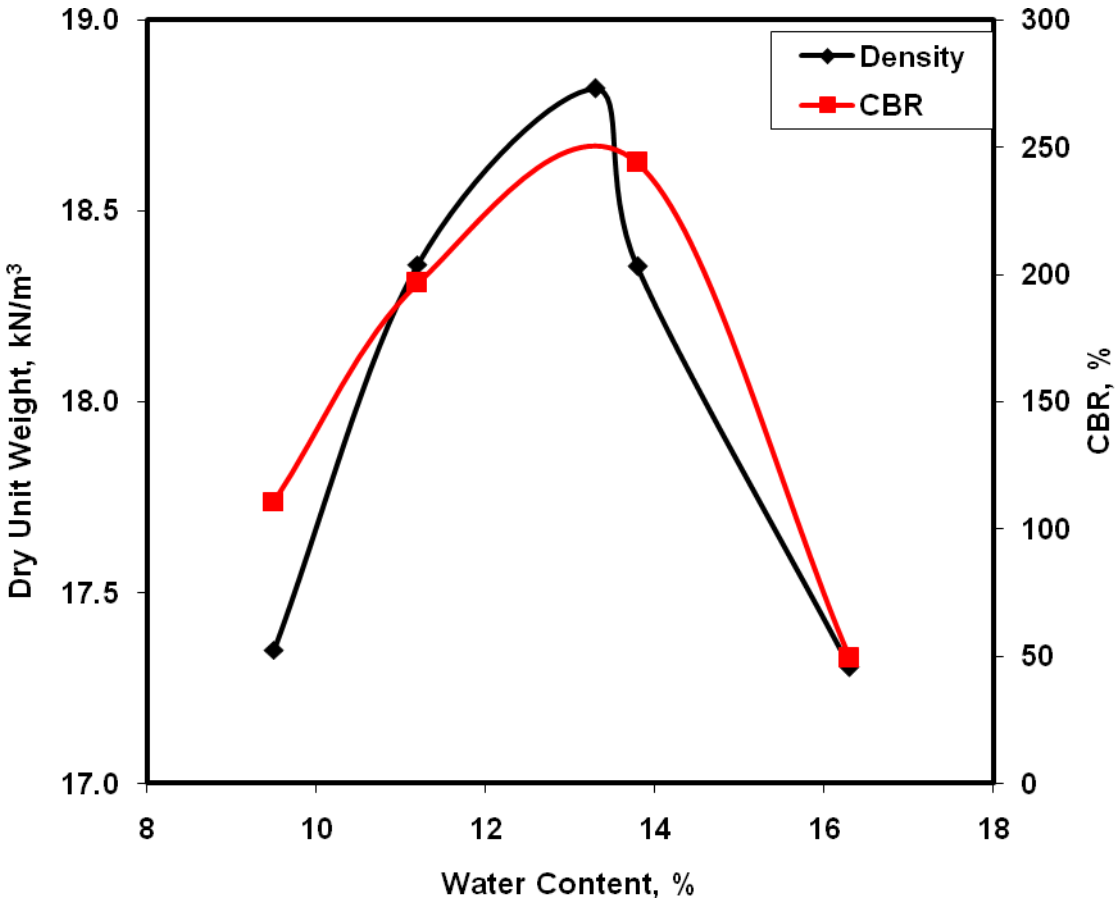

Figure 4-14: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 20% CKD Addition

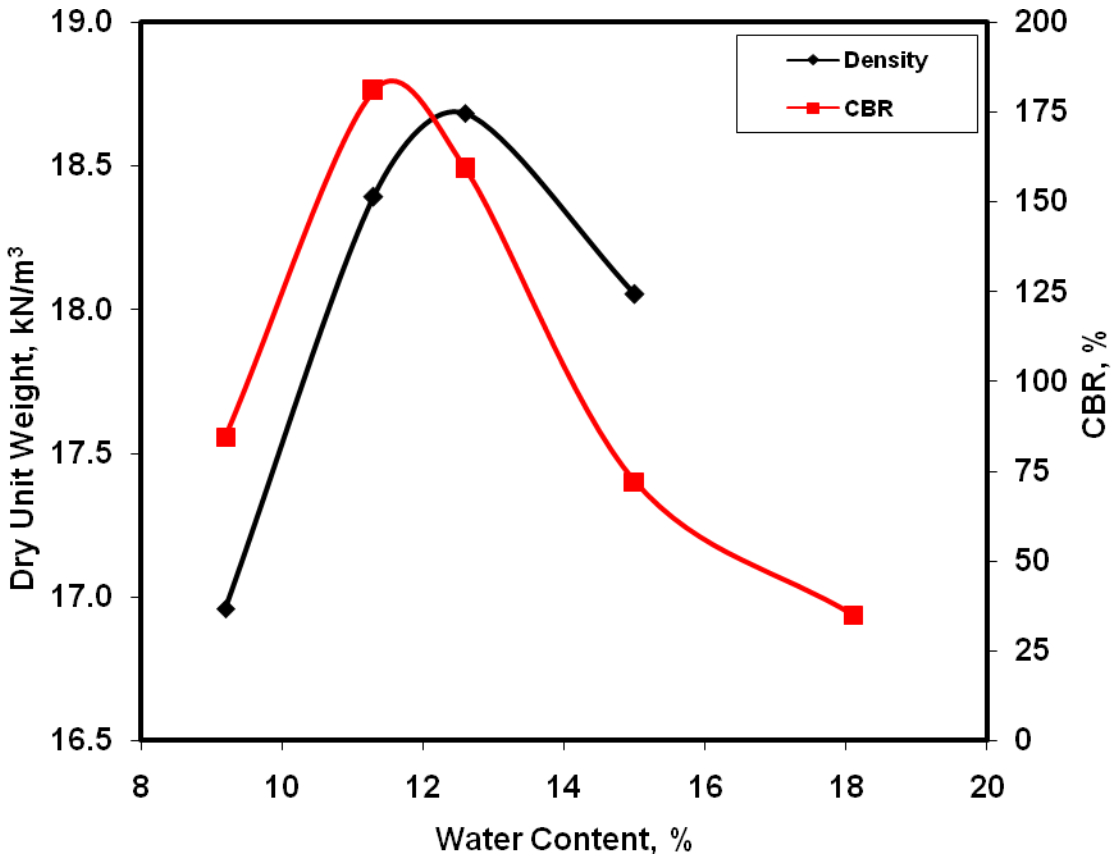

Figure 4-15: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 30% CKD Addition

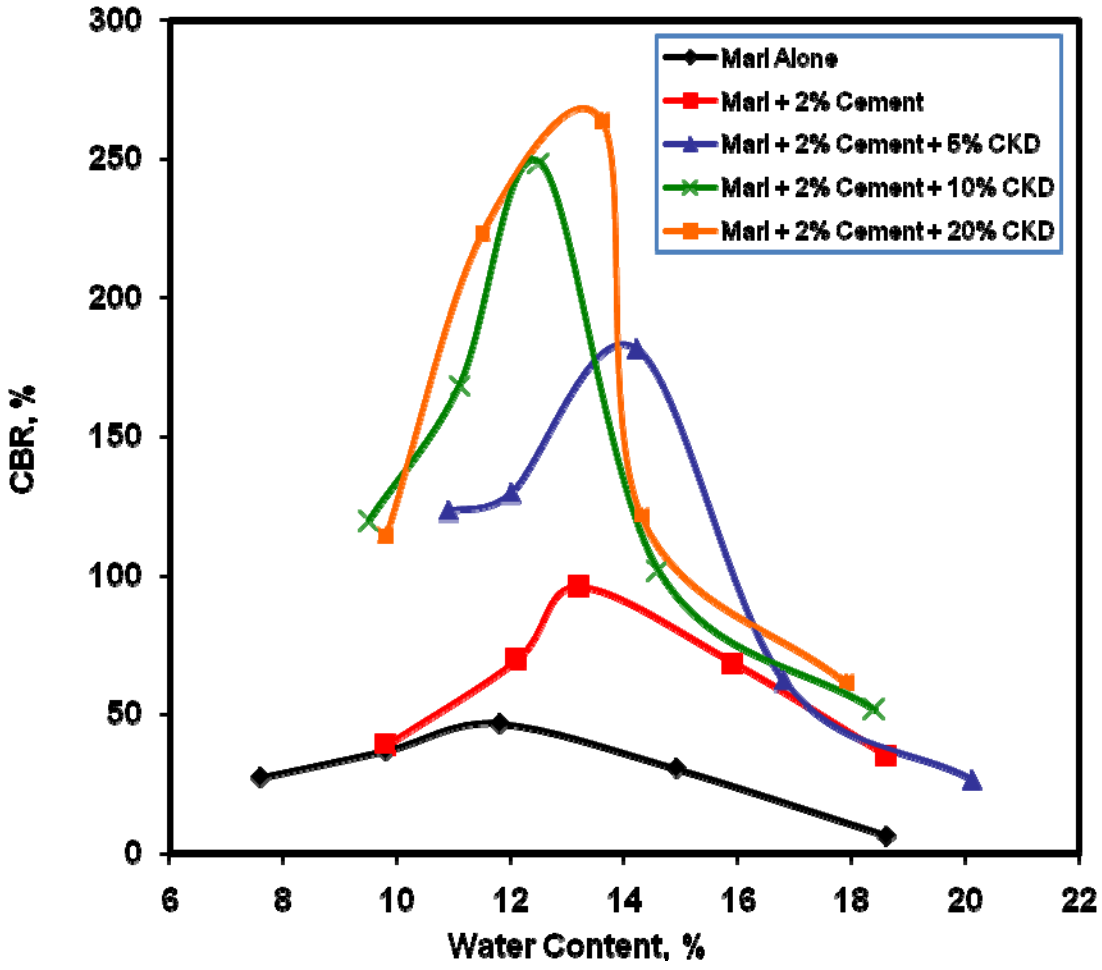

Figure 4-16: Effects of 2% Cement Addition with CKD Contents on CBR and Water Content Relationship of Non-Plastic Marl

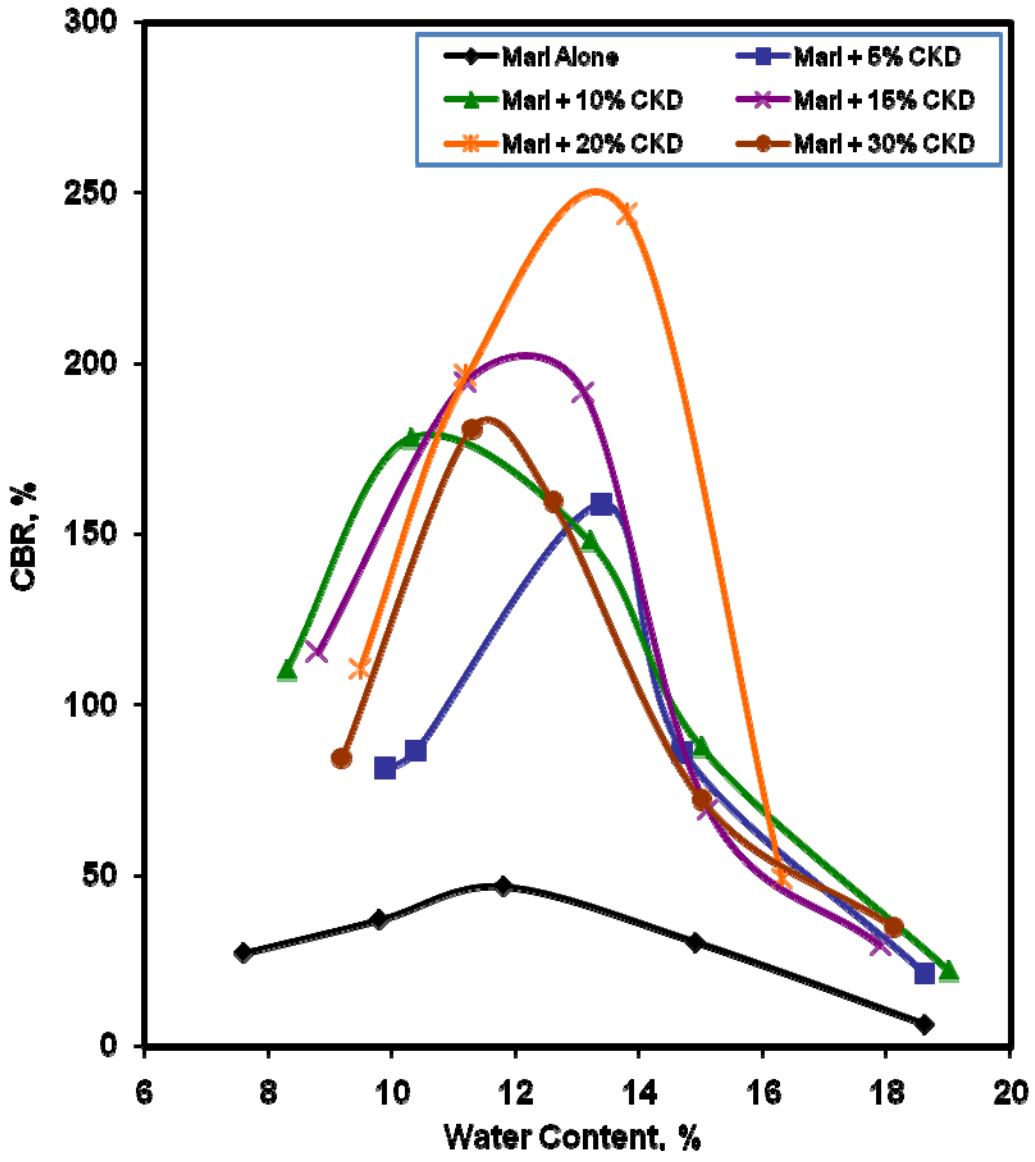

Figure 4-17: Effects of CKD Contents on CBR and Water Content Relationship of Non-Plastic Marl.

CBR value decreases when CKD content exceeds 20%. Similar trend is observed with the addition of 2% cement to the CKD content.

Figure 4.20 shows the relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 5% cement and 5% fuel fly ash (FFA). The maximum dry unit weight was 17.8 kN/m3 at an optimum moisture content of about 16.5%. On the other hand, the maximum CBR value was 164.5 at a moisture content of 14.9%. The CBR value increased initially with increasing the moisture content till the maximum value was attained at a moisture content less than that of optimum moisture content for the maximum dry unit weight. Any further increase in the moisture content led to a reduction in the CBR value. Comparing the curves in Figure 4.20 and the curves for untreated non-plastic marl and that treated with 5% cement in Figures 4.6 and 4.18, respectively, indicates that there was an increase in the CBR value. The maximum CBR value increased from 47 and 119 for plain marl and marl treated with 5% cement, respectively, to 164.5 with the addition of 5% FFA and 5% cement.

Table 4-3: Compaction and CBR Test Results for CKD-Non-Plastic Marl Soil

Cement (%)	CKD (%)	$(\gamma_d)_{max}$ kN/m^3	w_{opt} (%)	CBR (%)	w (%)
0	0	18.5	13	47	11.8
2	0	18	14	96	13.2
2	5	18.234	14.2	182	14.2
2	10	18.777	12.5	249	12.5
2	20	18.8	13	264	13
0	5	18.6	13.4	159	13.4
0	10	18.645	13.2	178	10.3
0	15	18.685	13.1	196	12
0	20	18.82	13.3	245	13.3
0	30	18.683	12.6	181	11.3

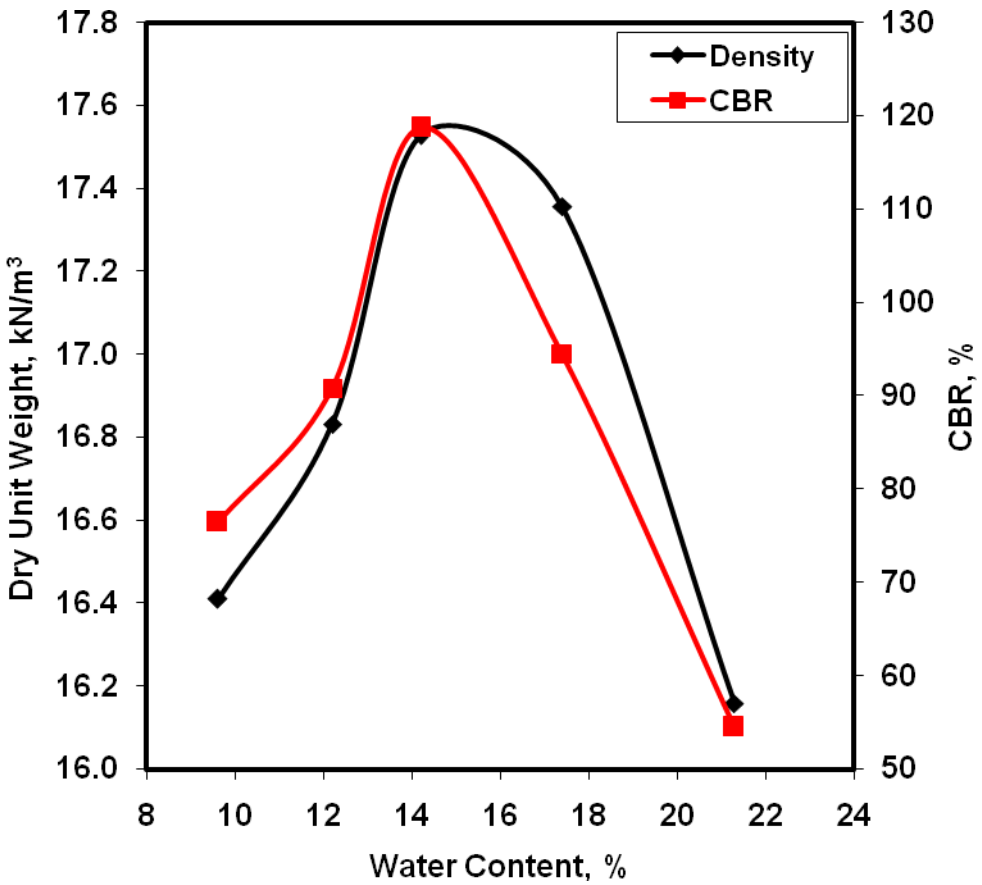

Figure 4-18: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% Cement Addition

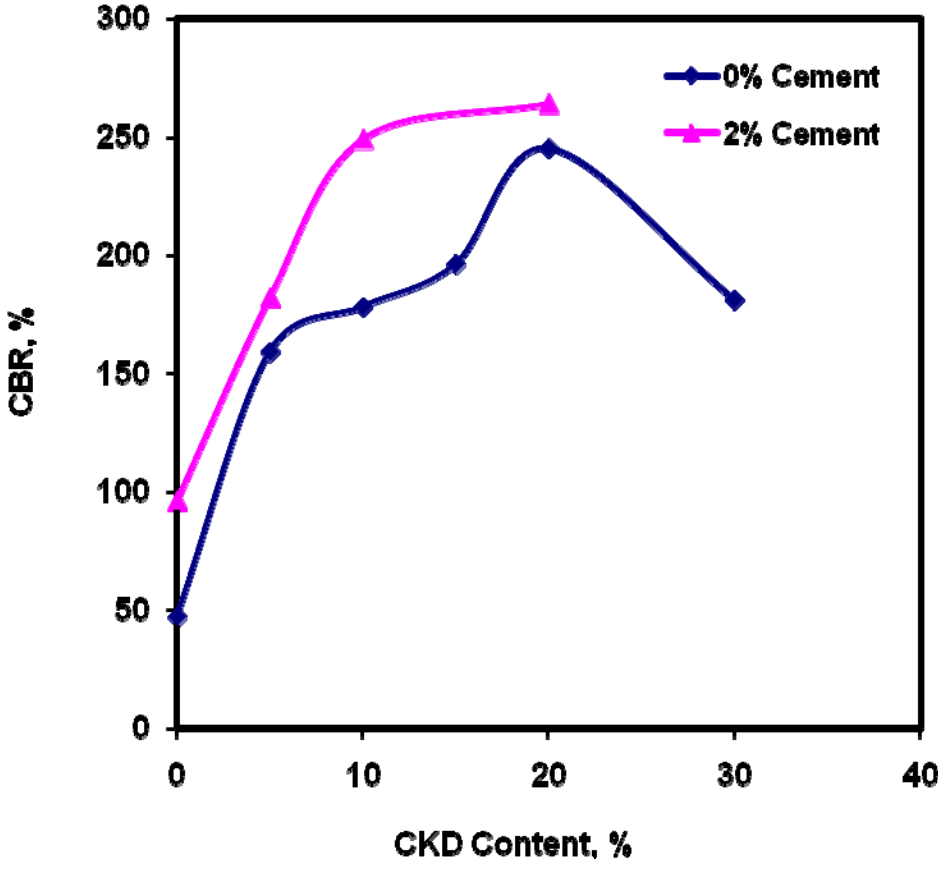

Figure 4-19: Maximum CBR Value-CKD Content Relationship for Non-Plastic Marl Soil

The relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 5% cement and 10% FFA is presented in Figure 4.21. It is seen that the maximum dry unit weight was 16.74 kN/m^3 at an optimum moisture content of 16.6% while the maximum CBR value was 158 at almost the same moisture content. Comparison of the data in Figure 4.21 and that in Figure 4.20 shows that there was a marginal reduction in the CBR value as well as in the dry density value when 5% cement and 10% FFA were added to the non-plastic marl. Furthermore, the moisture content increased with the increase in the FFA content. This is attributed to the fact that the FFA is a very fine material and has a high surface area which could absorb more volume of water.

Figure 4.22 depicts the relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 5% cement and 15% FFA. This figure shows that the maximum dry unit weight was 16.67 kN/m^3 at an optimum content of 19.1%. Similarly, the maximum CBR value was about 154 at a moisture content of about 18%. Comparison of the data in Figure 4.22 with that in Figure 4.21 indicates that there was a little reduction in the CBR value when 5% cement and 15% FFA were added to the non-plastic marl. Furthermore, there is an increase in the moisture content with increasing the ash content. The reduction in the CBR value could be attributed to the fact that FFA is not considered as a cementitious material. Further additions of FFA disrupt the granular structure of the non-plastic marl and cause the particles to float in the FFA. As a result, the dry density and the strength are reduced.

Figure 4.23 shows the relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 5% FFA alone (0% cement). It can be observed that the

maximum dry unit weight was 17.95 kN/m^3 at an optimum moisture content of 15.4%. On the other hand, the maximum CBR value was 116 at a moisture content of 13.4%. Comparison of the data in Figure 4.23 with that in Figure 4.6 of untreated non-plastic marl indicates that there was an increase in the CBR value. This increase was very high indicating that the FFA addition filled the voids between the marl particles and increased the strength. It can be seen also that there was an increase in the moisture content associated with the increase in FFA. Furthermore, the reduction in the dry unit weight was marginal.

Figure 4.24 presents the relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 10% FFA. The results from γ_d-w curve indicate that the maximum unit weight was 17.60 kN/m^3 at an optimum moisture content of 14.6%. For the CBR-moisture curve, however, the maximum CBR value was 105 at a moisture content of 13%. Comparison of the data in Figure 4.24 with that in Figure 4.23 indicates that there was a marginal decrease in the dry unit weight from 17.95 kN/m^3 to17.60 kN/m^3 and in the CBR value from 116 to 105. Similarly, there was a marginal decrease in the optimum moisture content from 15.4% to 14.6% when the 10% FFA was added.

The relationship of the moisture-unit weight-CBR for the stabilized non-plastic marl with 15% FFA is presented in Figure 4.25. It is seen from this figure that the maximum dry unit weight was 16.6 kN/m^3 at an optimum moisture content of 16.4%. For the CBR-moisture curve, the maximum CBR value was 91 at a moisture content of 16.4%. The maximum dry unit weight and the maximum CBR value were attained at the same moisture content. Comparison of the data in Figure 4.25 with that in Figure 4.24 indicates that there was a similarity in the shape of the CBR-w and γ_d-w curves. In

addition, there was a decrease in the maximum dry unit weight as well as in the maximum CBR value when 15% FFA was added to the non-plastic marl soil. The maximum unit weight and the maximum CBR value decreased from 17.60 kN/m^3 and 105 to 16.6 kN/m^3 and 91, respectively. However, there was an increase in the optimum moisture content to16.4%.

Figure 4.26, Figure 4.27 and Table 4.4 summarize the CBR test results for FFA stabilized- non-plastic marl mixtures. From Figure 4.26 it is noted that the maximum CBR value for the marl soil (0%) was 47. Thereafter, this value increased to about 119 due to the addition of 5% cement. This is attributed to the fact that cement tends to bond the soil particles and leads to the increase in strength. From the data in the two figures, it is clear that as the addition of fuel fly ash (FFA) increased beyond 5%, the CBR decreased and the optimum moisture content increased. However, all the ash-based samples had much higher CBR values as compared with the plain non-plastic marl samples. The data in Figure 4.26 indicates that the CBR value increased from 47 for 0% addition (untreated non-plastic marl) to 165 for 5% FFA and 5% cement additions and then decreased to 158 and 149 when 10 and 15% FFA with 5% cement additions were added to the non-plastic marl, respectively. It can be seen that such a reduction in the CBR value was marginal. The data in Figure 4.27 also indicates that the CBR value increased form 47 for untreated non-plastic marl (0% addition) to 116 when 5% FFA was added. Thereafter, it decreased to 105 and 91 when 10 and 15% FFA were added to the marl soil. Again, the reduction in the CBR value was marginal and still much higher than that for the untreated soil.

From the previous results, it can be concluded that the soils compacted on the dry side of the optimum moisture content bear higher strength than those compacted on the wet side of optimum. This is ascribed to the fact that the formation of the large-sized strong clods (macropeds) on the dry side provides high frictional resistance and to the small increase in the effective stresses due to suction. Furthermore, the dry cohesive samples had higher strength compared to those with higher moisture content. On the wet side of optimum, the macropeds got smaller and weaker causing a reduction in the cohesion and thus in the strength. However, the higher density at the optimum moisture content and the intermediate macropores and macropeds results in a higher strength which is partially attributed to the relatively low moisture content [Ahmed, 1995].

This behavior of marl soils on the dry side of optimum may be attributed to the absence of cohesive material and large macropores and, therefore, the macropeds become friable, unstable and relatively weak. When the moisture content decreases below the optimum, the dry density decreases which in turn causes a decrease in the interlocking. However, at moisture content at or near $w_{opt,}$ denser macropeds give high interlocking, which is responsible for the strong and stable soil mass. Since there are no cohesive fines in the soil, the strength is derived only through partial interlocking [Ahmed, 1995].

Similarly on the wet side of optimum, the non-cohesive carbonate fines from the loose lumps and excess water result in a loss of cementation near the contact points. The soil mass becomes mud-like lumps with no bearing strength. The large-sized aggregate will just float in the loose matrix (lumps) of the fine particles [Ahmed, 1995].

Figure 4.28 presents the relationship of the maximum CBR value-FFA content for the non-plastic marl soil. It is seen that the maximum CBR value was attained at 5% FFA

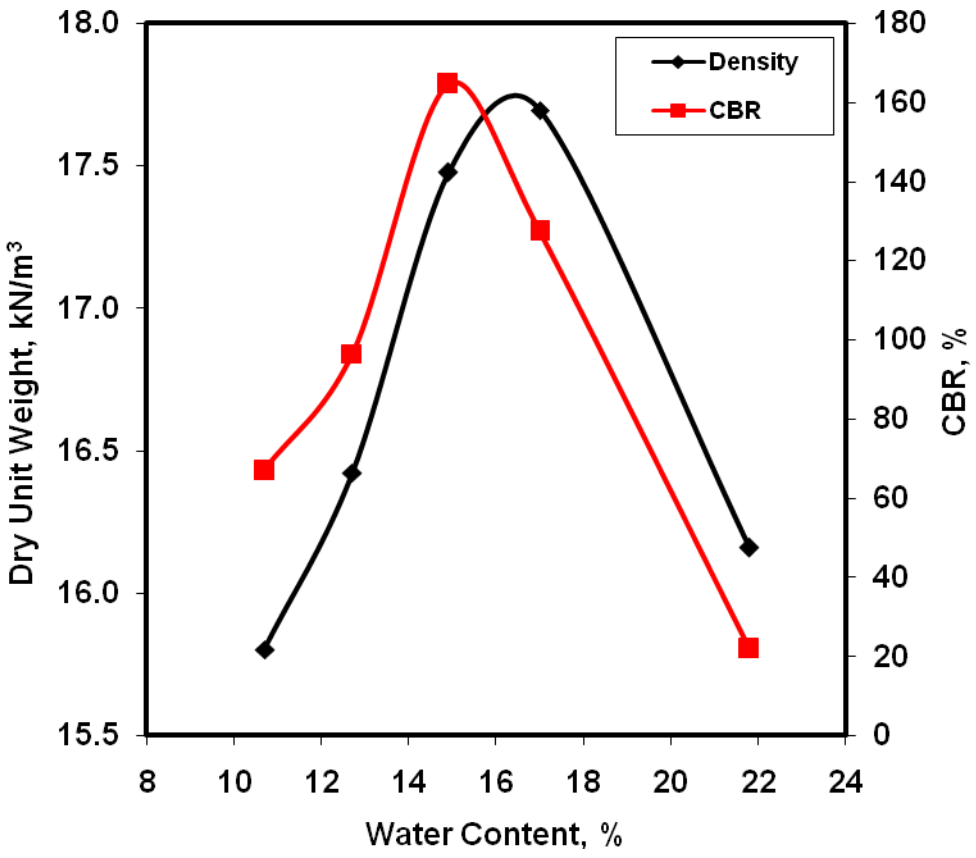

Figure 4-20: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% Cement and 5% FFA Additions

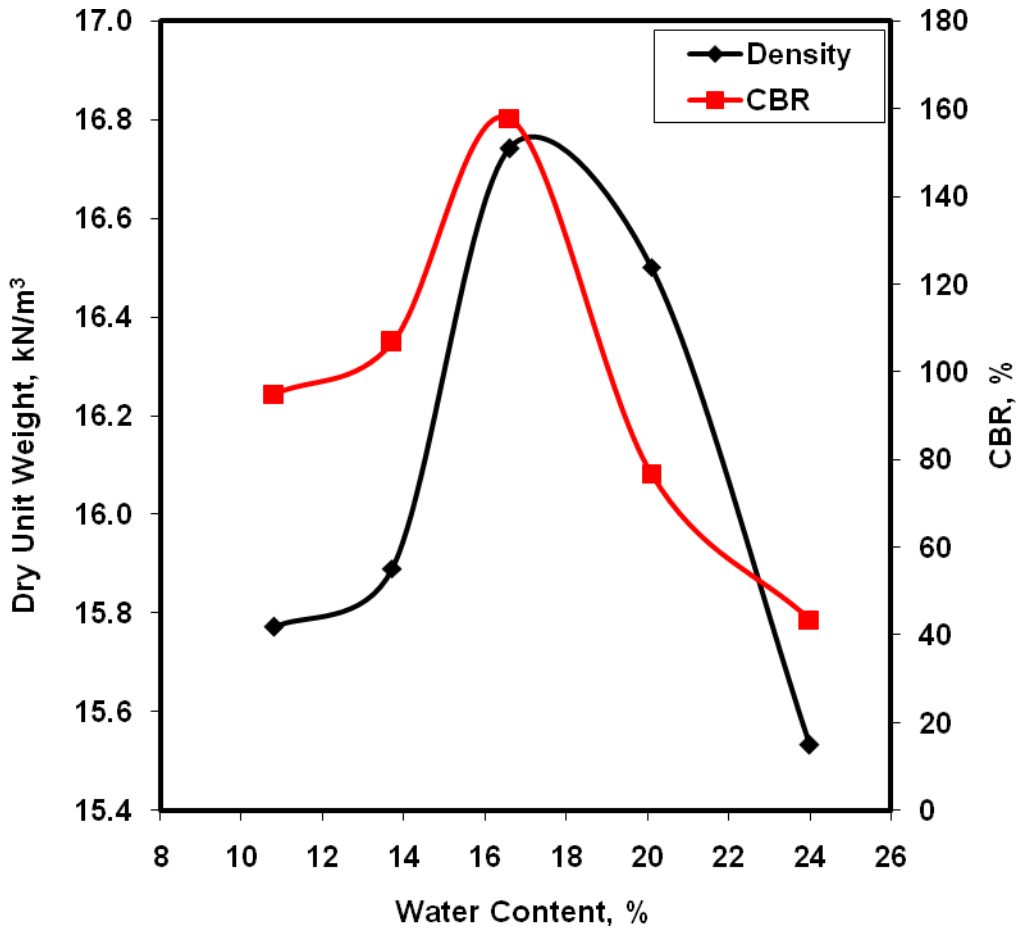

Figure 4-21: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% Cement and 10% FFA Additions

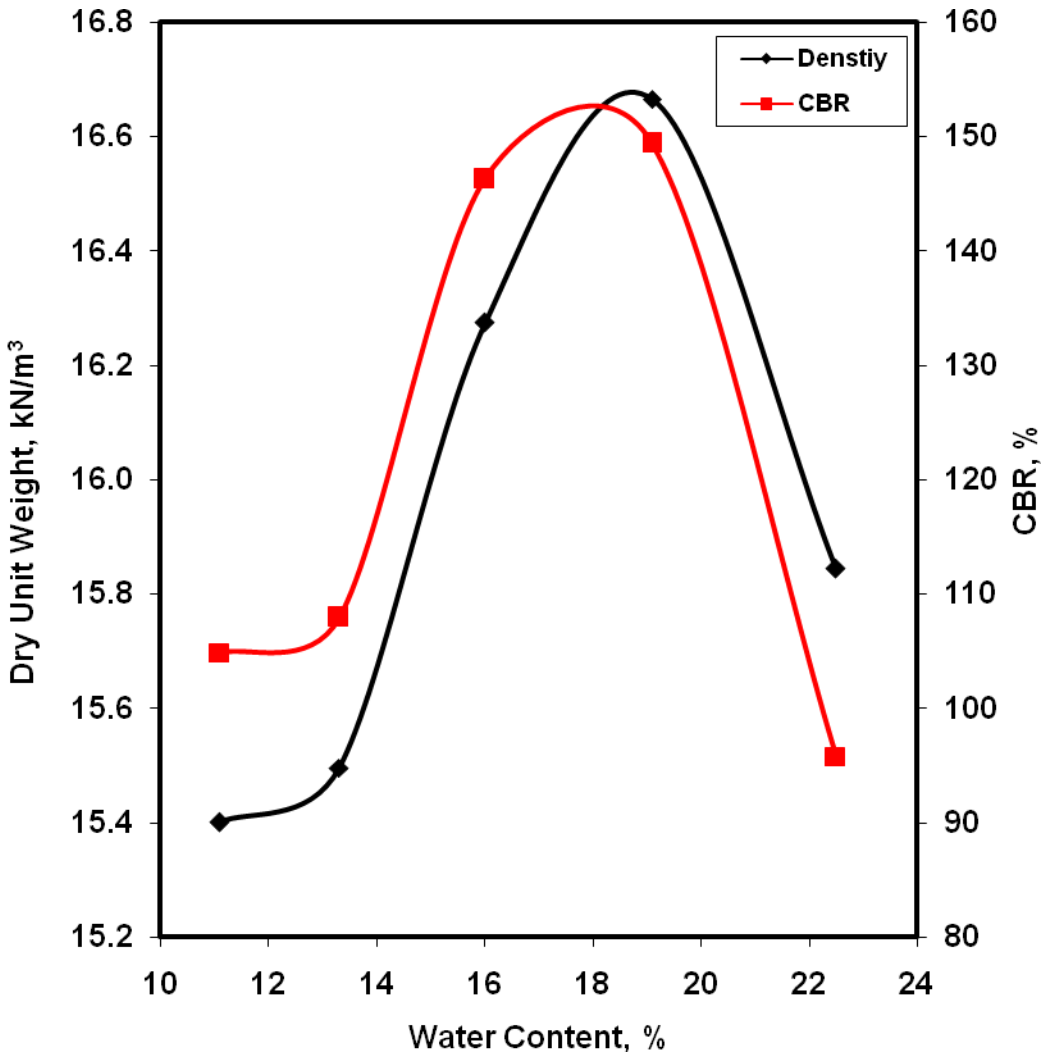

Figure 4-22: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% Cement and 15% FFA Additions

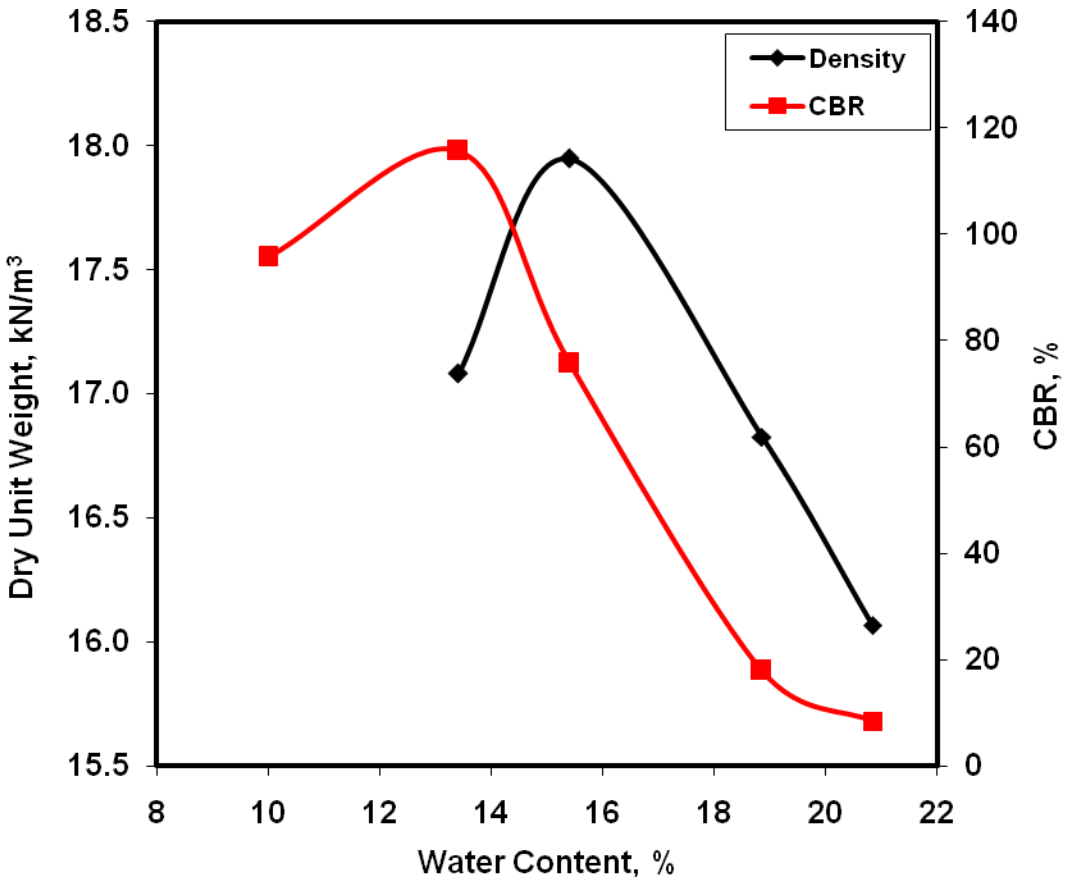

Figure 4-23: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 5% FFA Addition

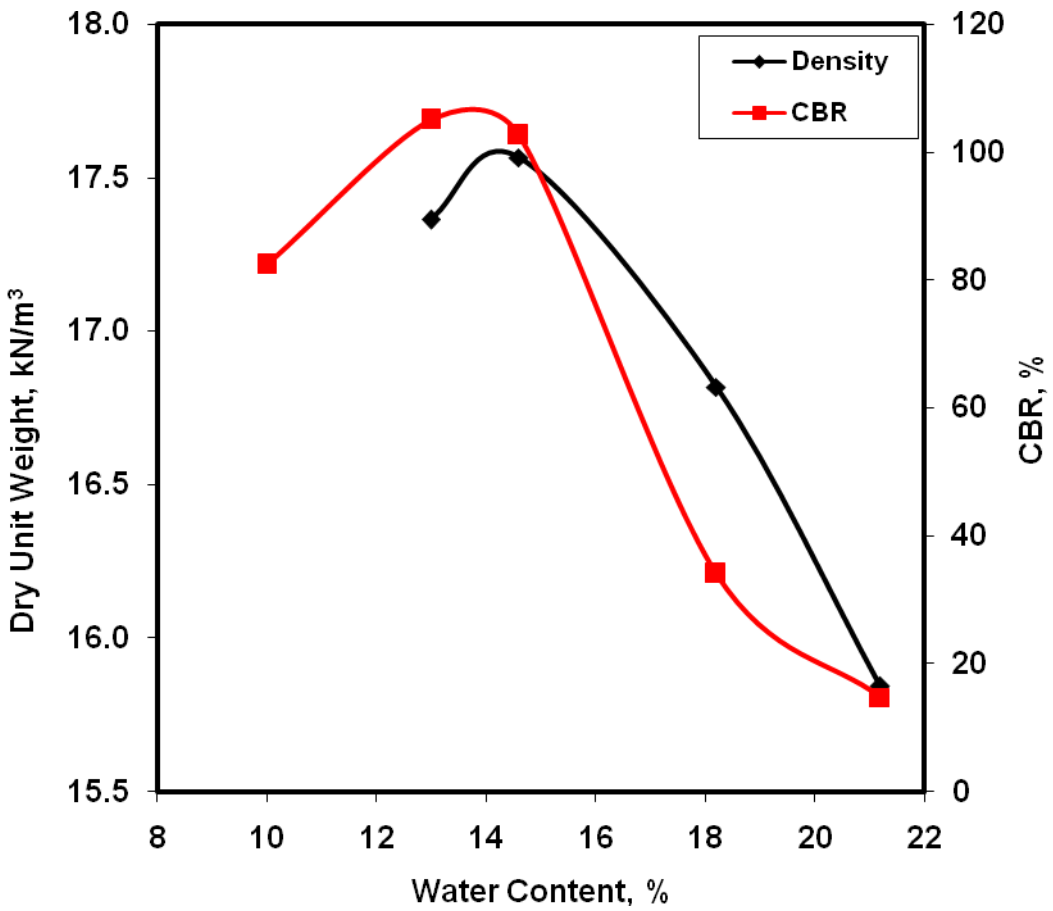

Figure 4-24: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 10% FFA Addition

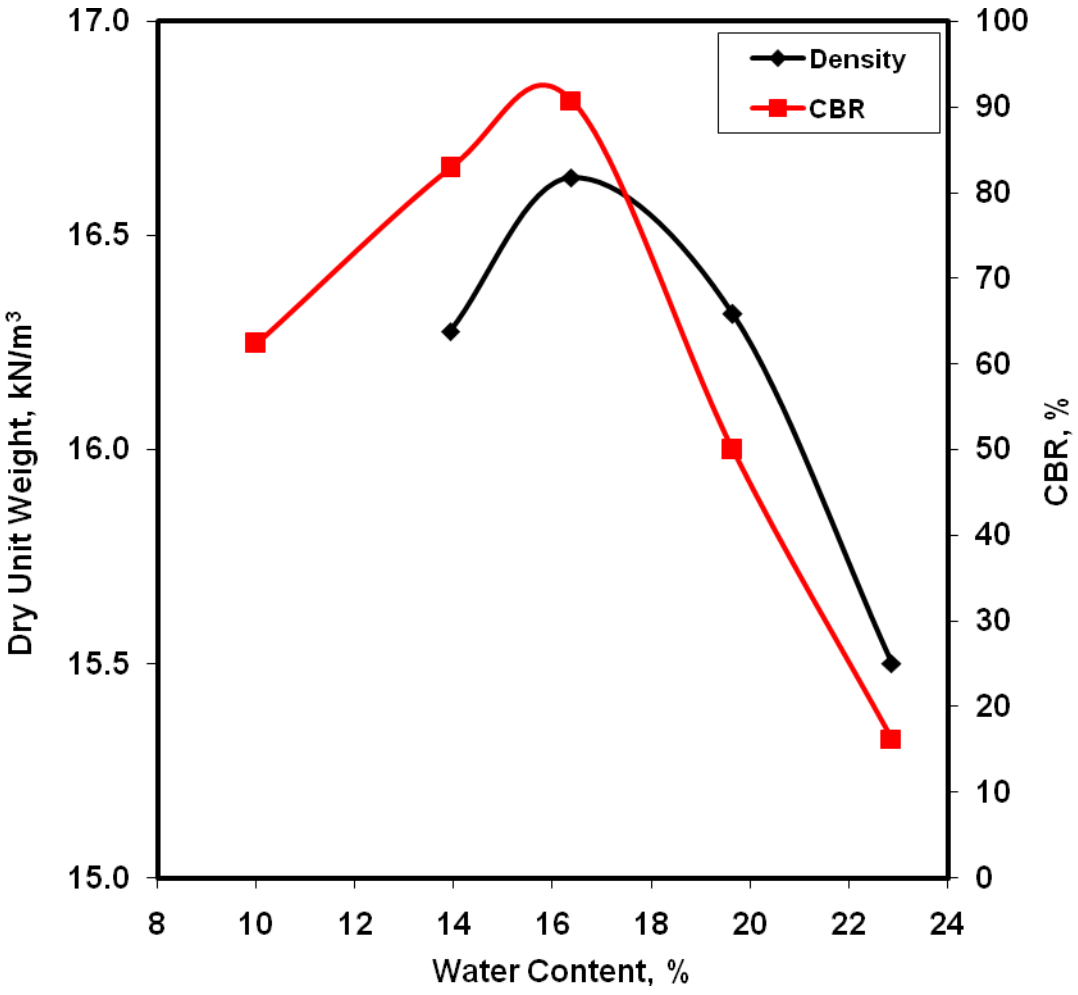

Figure 4-25: Moisture-Unit Weight-CBR Relationship for Non-Plastic Marl Soil with 15% FFA Addition

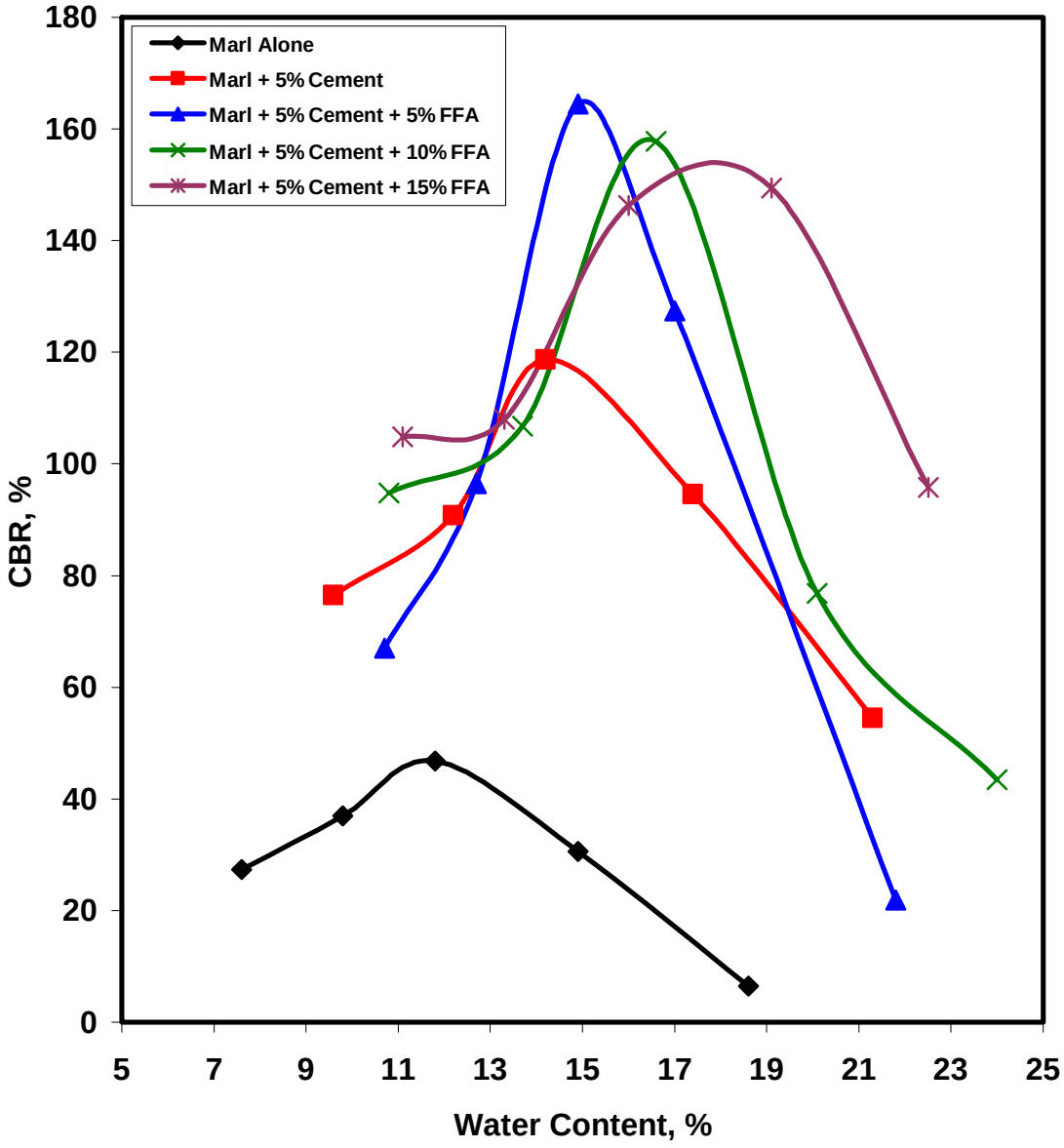

Figure 4-26: Effects of Moisture and 5% Cement with FFA Contents on CBR of Non-Plastic Marl

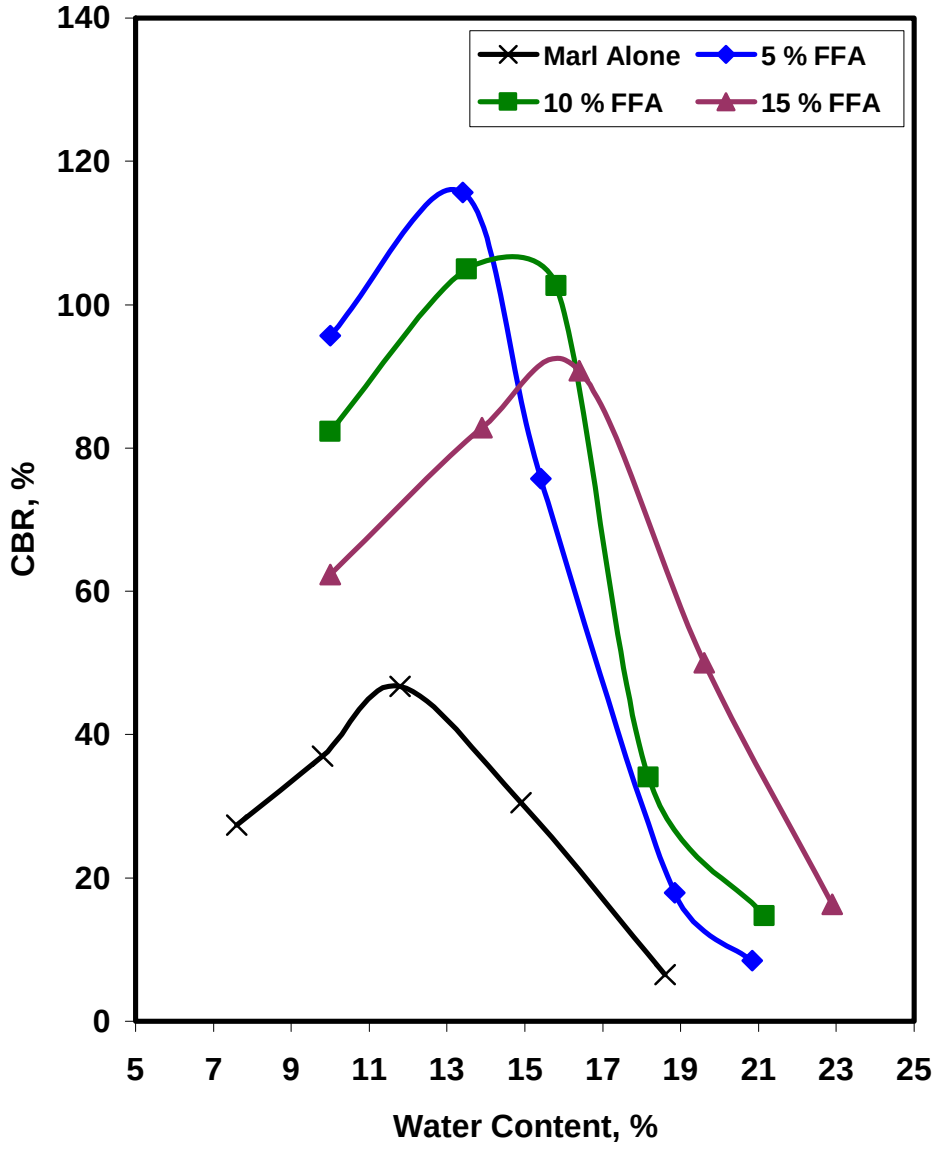

Figure 4-27: Effects of Moisture and FFA Contents on CBR of Non-Plastic Marl

plus 5% cement. From the same figure, it is also observed that further increase in FFA addition beyond 5% led to a gradual decrease in the maximum CBR value. This is could be ascribed to the overdose and lubrication effects which tend to reduce the strength.

Table 4-4: Compaction and CBR Test Results for FFA-Non-Plastic Marl Soil

Cement (%)	FFA (%)	$(\gamma_d)_{max}$ kN/m^3	w_{opt} (%)	CBR (%)	w (%)
0	0	18.5	13	47	11.8
5	0	17.528	14.2	119	14.2
5	5	17.75	16.5	164.5	14.9
5	10	16.74	16.6	158	16.6
5	15	16.666	19.1	154	18
0	5	17.949	15.4	116	13.4
0	10	17.6	14.6	105	13
0	15	16.635	16.4	91	16.4

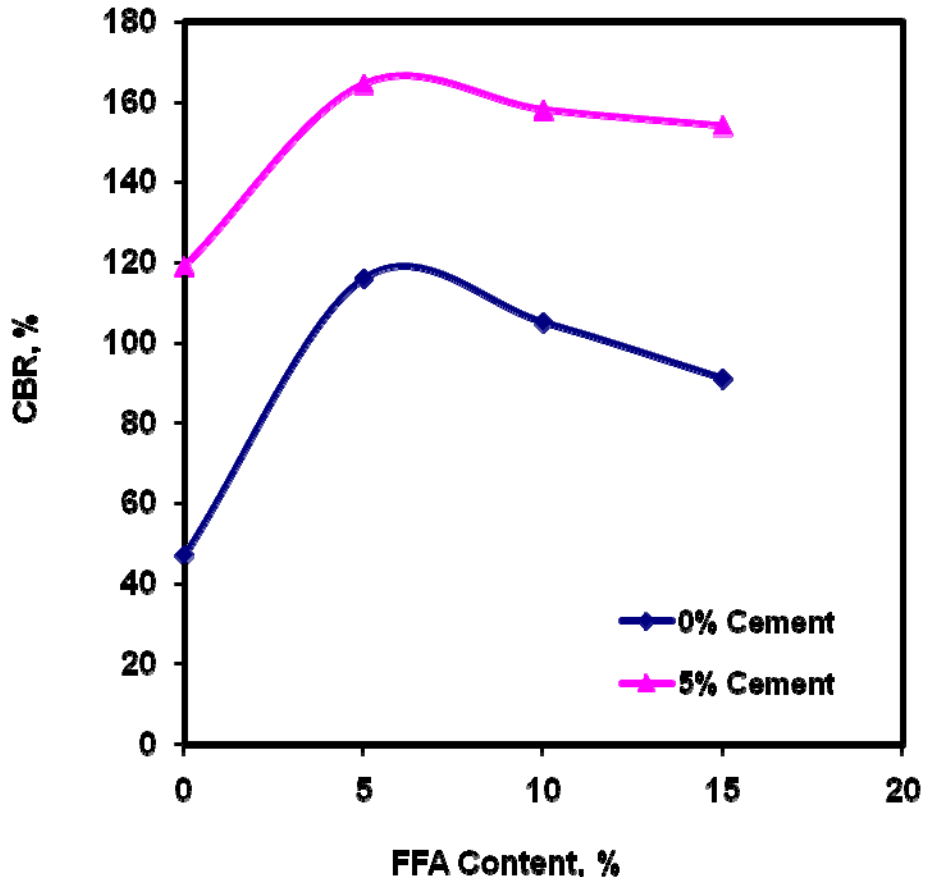

Figure 4-28: Maximum CBR Value-FFA Content Relationship for Non-Plastic Marl Soil

4.3.3 Unconfined Compressive Test Results

This test was adopted as a basic technique to study the effect of age and additive content on strength gain of the treated mixes. In this test, all samples were compacted at the optimum moisture content in a mold with a height (h) to diameter (d) ratio of 2 (d = 50 mm, h = 100 mm). Only sealed regime was adopted for curing and all samples were cured at laboratory condition (23 ± 3°C).

4.3.3.1 CKD-Marl Mixtures

The non-plastic marl soil was treated with different percentages of CKD and cement and CKD alone as listed in Table 3.2. The effect of age and CKD content on the strength gain is discussed thoroughly in the following sections.

a) Effect of curing time

All samples were prepared and tested after curing periods of 3, 7, 14, and 28 days at a laboratory temperature (23 ± 3°C). The variation of unconfined compressive strength (q_u) of CKD-non-plastic marl mixtures with curing period is presented in Figures 4.29 and 4.30. The results clearly indicate that there was an approximately linear relationship between the q_u and curing period for all the data in Figures 4.29 and 4.30 and the rate of strength gain was slower in the initial days of curing. It can also be seen that the strength seemed to increase with time, even beyond 28 days. This continued gaining strength could be ascribed to the availability of sufficient moisture content during the course of curing since the samples were sealed.

Knowledge of the variation in strength gain with time is important in design and construction procedures. It is used in the decision making for the curing methods and periods. Usually 7 day q_u is used for design purposes [Bhatia, 1967], while 3 day strength

is used for quality control purposes in field construction. Strength after 28 days can also be used for the capacity evaluation of an existing pavement. However, to speed up construction, the 24-hour strength is sometimes utilized.

The reason for the increase in strength is attributed to the fact that cement and CKD usually need some period to develop their strength and as the curing period increases, the cement-CKD hydration products will increase leading to an increase in strength.

b) Effect of additive content

The relationship between the unconfined compressive strength (q_u) and CKD content is presented in Figures 4.31 and 4.32. It is seen that q_u increased significantly with increasing the CKD and cement content in approximately linear relationship. Furthermore, there was a sharp increase in the q_u for the mixtures with CKD alone (Figure 4.32) with increasing the CKD content until a dosage of 20% by dry weight of the soil and thereafter the increase was marginal. It can also be noted that the non-plastic marl treated with combined stabilizer (CKD + cement) has developed much higher strength than those stabilized with CKD alone.

According to the ACI Committee 230 Report (ACI, 1990), the minimum 7-day q_u specified for subbase and subgrade in rigid pavement construction by the USA Army Corps of Engineers (USACE) is 1,380 kPa (200 psi), while for base course it is 3,450 kPa (500 psi). For flexible pavement construction, however, these values are 1,725 kPa (250 psi) and 5,175 kPa (750 psi), respectively.

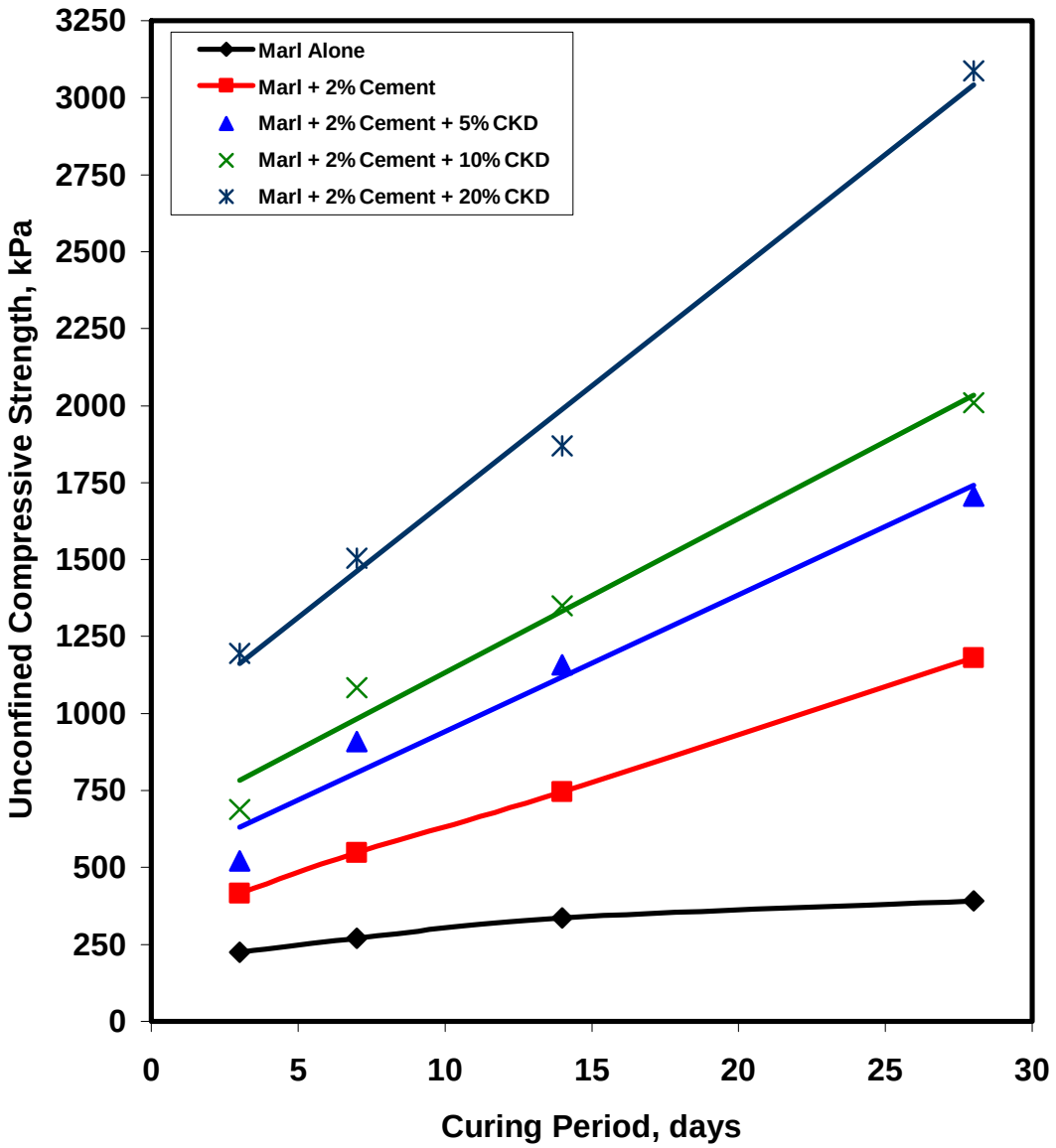

Figure 4-29: Variation of the q_u with Curing Period for CKD-Cement-Non-plastic Marl Mixtures

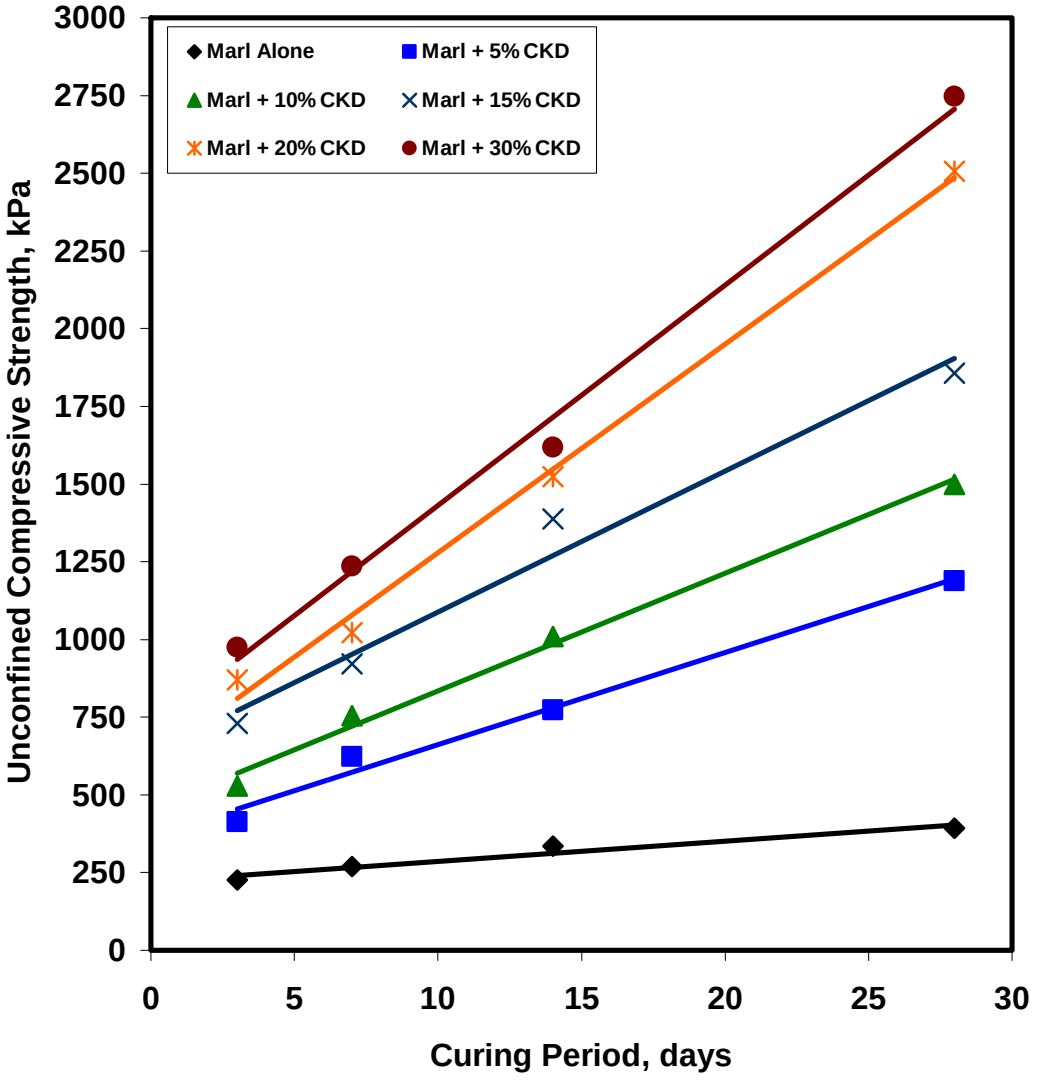

Figure 4-30: Variation of the q_u with Curing Period for CKD-Non-plastic Marl Mixtures

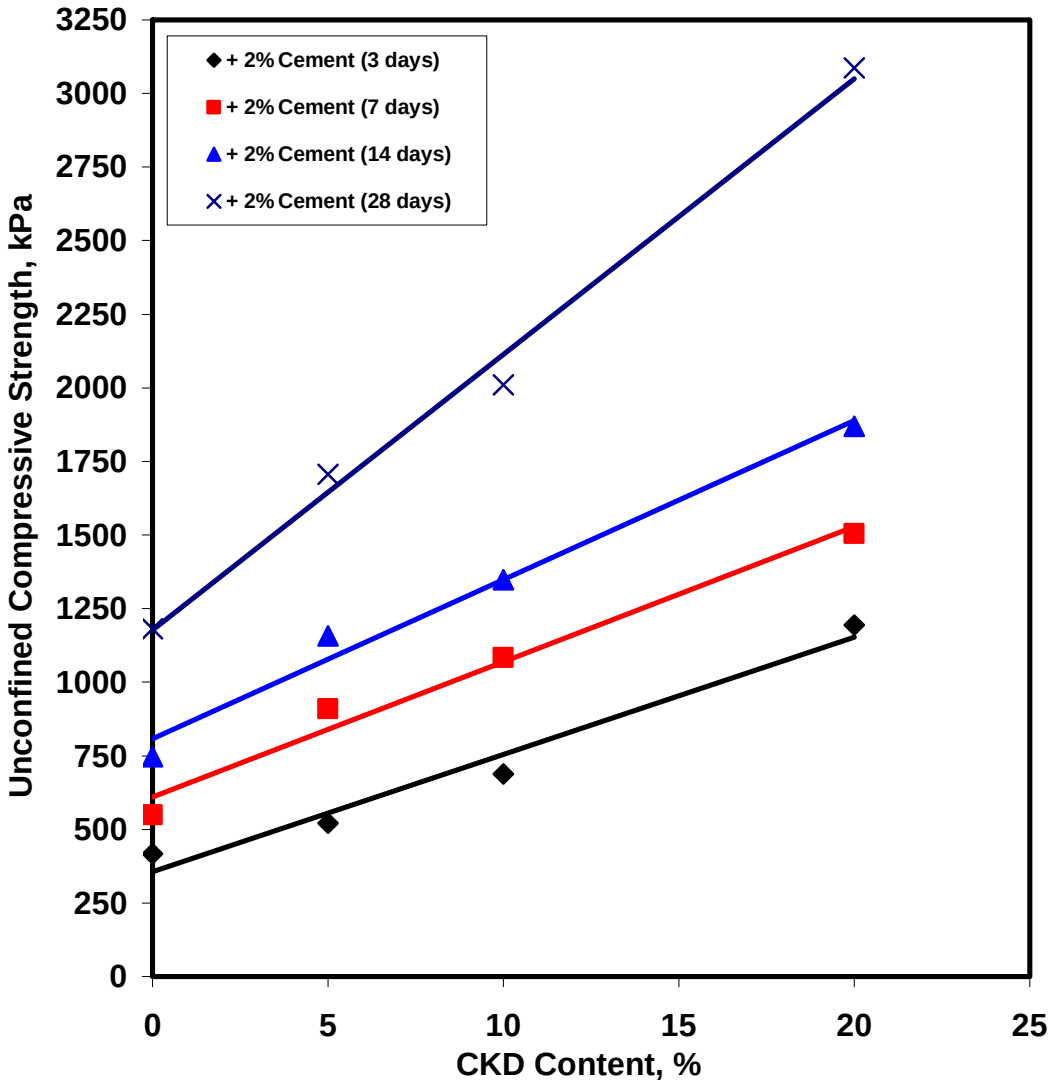

Figure 4-31: Variation of the q_u with CKD content for CKD-Cement-Non-plastic Marl Mixtures

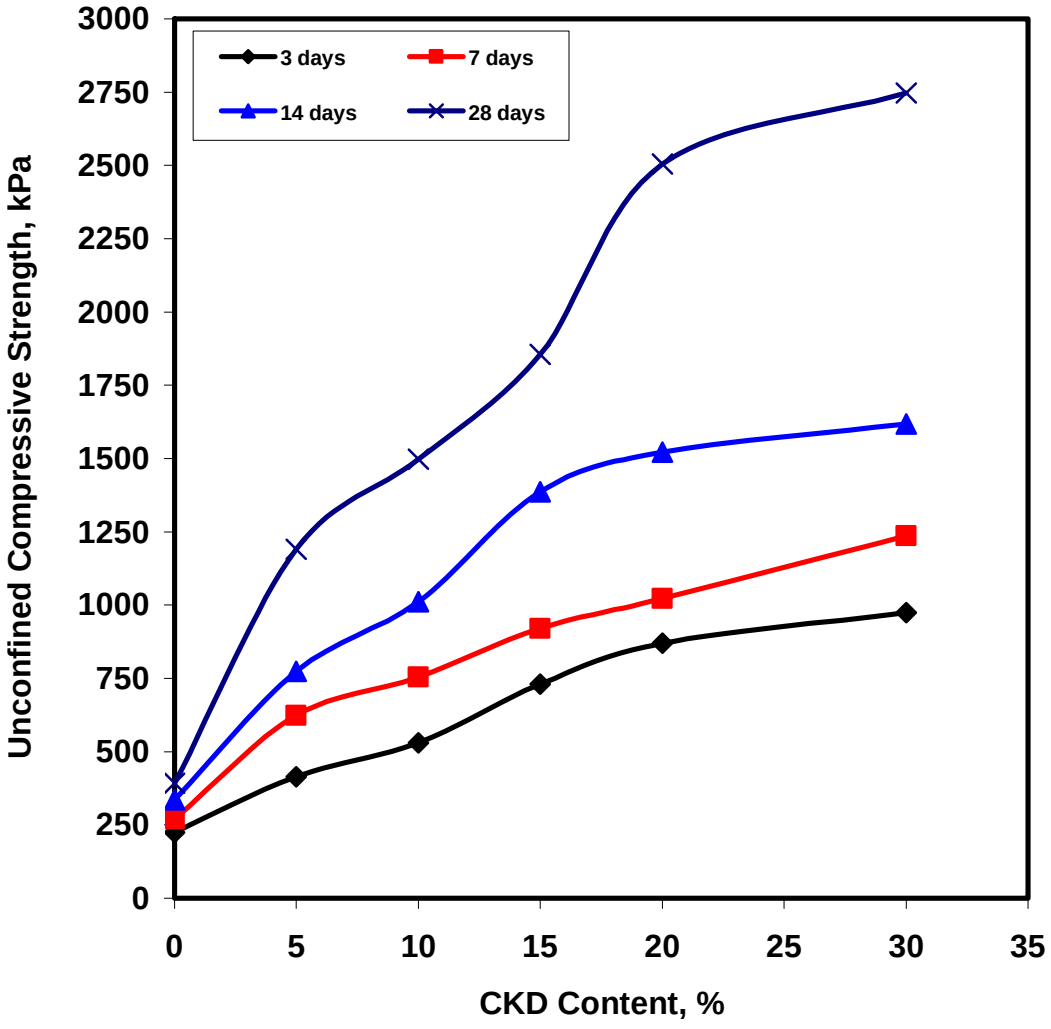

Figure 4-32: Variation of the q_u with CKD content for CKD-Non-plastic Marl Mixtures

Results of the present investigation (Table 4.5) indicate that only the marl soil treated with 2% cement + 20 % CKD satisfied the 7-day strength requirements according to the ACI Committee 230 Report (ACI, 1990).

Ahmad [1995] indicated that a cement content of 5% would be enough for the effective stabilization of local calcareous soils (i.e. marl soil) and met the strength and durability requirements. In the present investigation, the cement content was limited to only 2% in order to encourage the usage of CKD waste material. However, it seems that the usage of CKD in content up to 30% would not satisfy the above-referenced ACI requirements.

Table 4-5: Unconfined Compressive Strength Test Results for CKD-Marl Soil

Cement (%)	CKD (%)	Unconfined Compressive Strength (kPa)			
		3 days	7 days	14 days	28 days
0	0	225.55	269.51	336.05	392.53
2	0	415.82	548.79	745.99	1181.42
2	5	521.43	909.77	1158.02	1706.80
2	10	689.01	1083.73	1349.01	2009.91
2	20	1194.28	1503.73	1868.78	3087.24
0	5	413.41	623.73	773.63	1189.45
0	10	529.45	754.29	1010.54	1498.89
0	15	730.11	921.10	1387.69	1856.04
0	20	870.33	1021.86	1522.68	2506.14
0	30	975.00	1235.38	1617.24	2746.36

4.3.3.2 FFA-Marl Mixtures

Various percentages of fuel fly ash (FFA) with 5% cement or FFA alone as listed in the Table 3.2 were used to treat the non-plastic marl soil. All samples were compacted at the optimum moisture content in a mold of 50 mm diameter and 100 mm height (h/d = 2). Thereafter, the samples were sealed and left to cure at the laboratory condition (23 ± 3°C) until testing. The effect of curing period and FFA content on the strength gain is presented in the following sections.

a) Effect of curing period

The effect of curing period on strength of non-plastic marl-FFA mixtures was studied. Sealed samples were prepared at the optimum moisture content and tested after curing periods of 3, 7, 14, and 28 days at the laboratory temperature (23 ± 3°C).

Figure 4.33 and Figure 4.34 present the relationship between the unconfined compressive strength (q_u) and curing period. The results in Figure 4.33 clearly indicate that the strength of treated marl with 5% cement and various percentages of FFA increased with the extended period of curing. It is seen that the relationship between q_u and curing period was nearly linear and there was a continued strength gain during the tried period. This is attributed to the availability of sufficient moisture content for hydration since the specimens are sealed, thus, there is no moisture loss. Similarly, Figure 4.34 shows the same trend for marl treated with FFA only. However, the unconfined compressive strength is much higher when 5% cement was used with FFA.

b) Effect of additive content

The relationship between the unconfined compressive strength and additive content for the FFA-cement-non-plastic marl mixtures and FFA-non-plastic marl are

presented in the Figure 4.35 and Figure 4.36, respectively. It is noticed that there was a sharp increase in the strength when 5% FFA and 5% cement were added to the non-plastic marl soil, thereafter, further increase in the FFA content led to the gradual decrease in the unconfined compressive strength.

The sharp increase, as shown in Figure 4.35, in the unconfined compressive strength when 5% FFA and 5% cement additions were used indicates that the additives filled the voids of the untreated non-plastic marl and made it so denser. Similarly, when the FFA alone was added to the parent soil (untreated non-plastic marl), the q_u increased sharply due to the addition 5% FFA and, thereafter, decreased gradually with increasing the FFA content, as shown in Figure 4.36. It is seen that the unconfined compressive strength is much higher when 5% cement was used with the various percentages of FFA. Results in all figures and Table 4.6 clearly indicate that only non-plastic marl stabilized with 5% cement and 5% FFA having an unconfined compressive strength of 1386.3 kPa satisfied the 7-day strength requirements according to the ACI Committee 230 Report (ACI, 1990). However, the other mixes (cement with FFA) gave q_u close to the 7-day strength requirements according to the ACI Committee 230 Report (ACI, 1990). Furthermore, other cement and FFA mixtures can be used in many other engineering applications such as improving the bearing capacity.

It is well mentioning that FFA may have hazardous ingredients such as heavy metal (vanadium and nickel) which are deleterious to the ground water and the environmental at large. Therefore, caution has to be practice to warranty that there will be no "bad" impact on the environment.

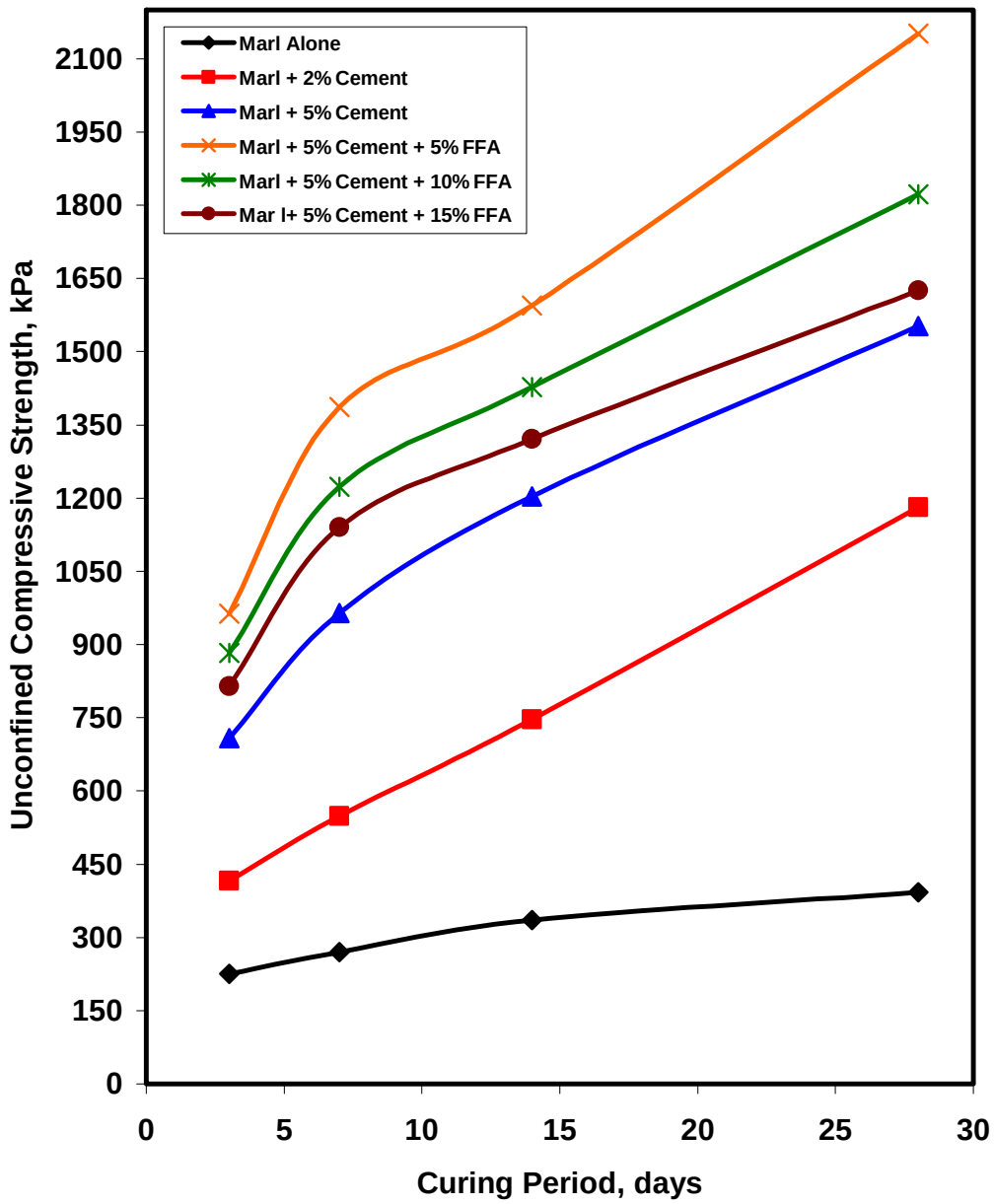

Figure 4-33: Variation of the q_u with Curing Period for FFA-Cement-Non-plastic Marl Mixtures

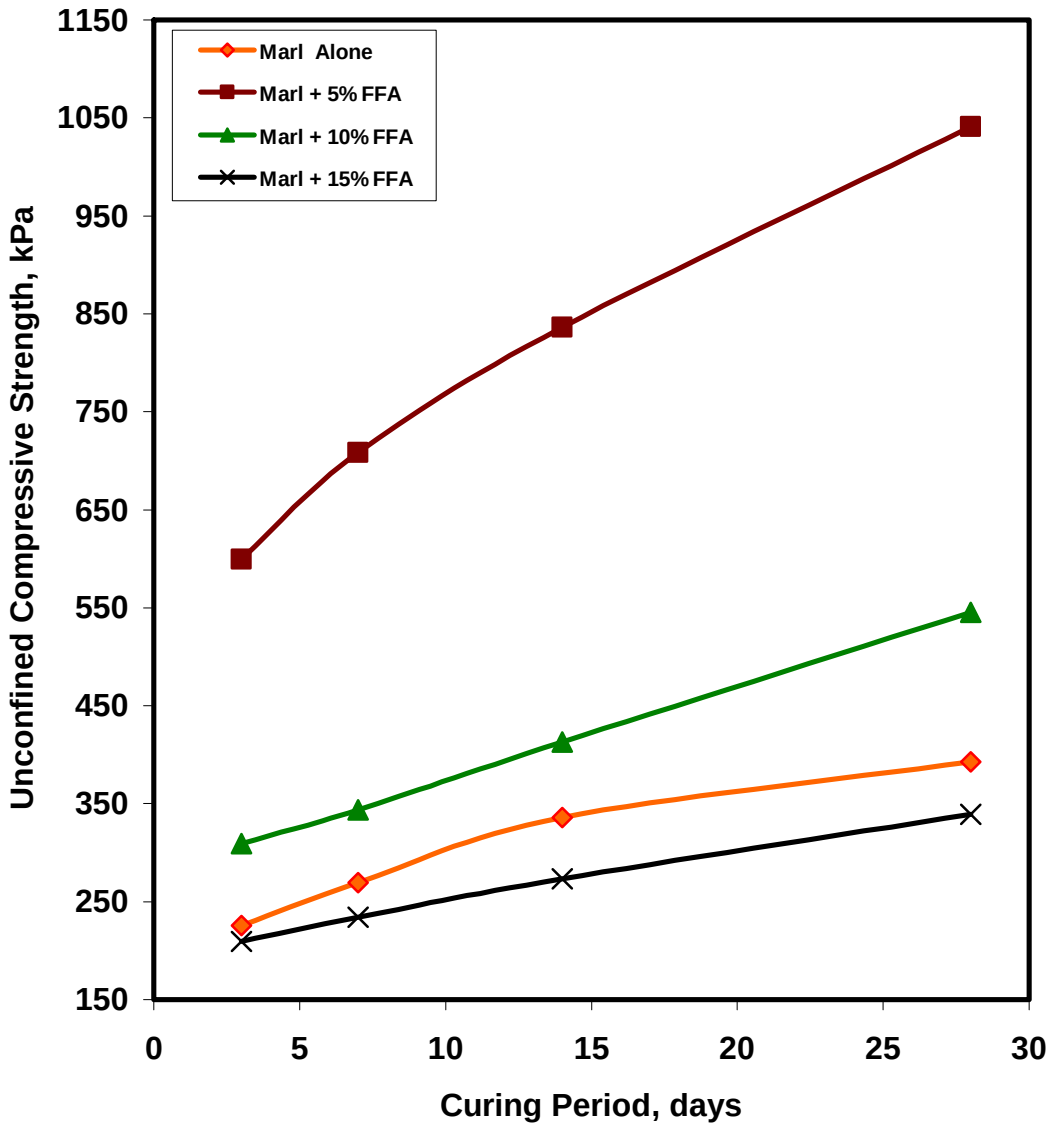

Figure 4-34: Variation of the q_u with Curing Period for FFA-Non-plastic Marl Mixtures

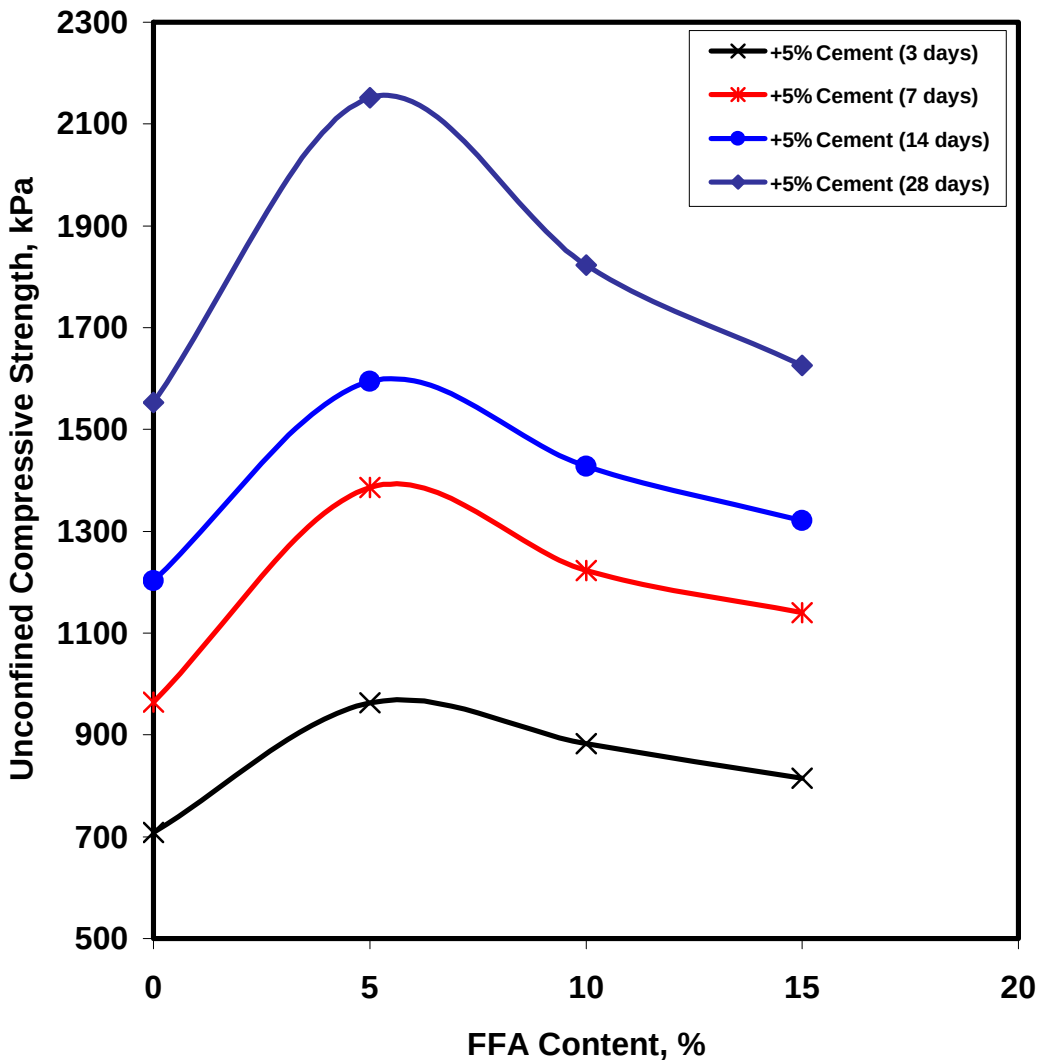

Figure 4-35: Variation of the q_u with FFA content for FFA-Cement-Non-plastic Marl Mixtures

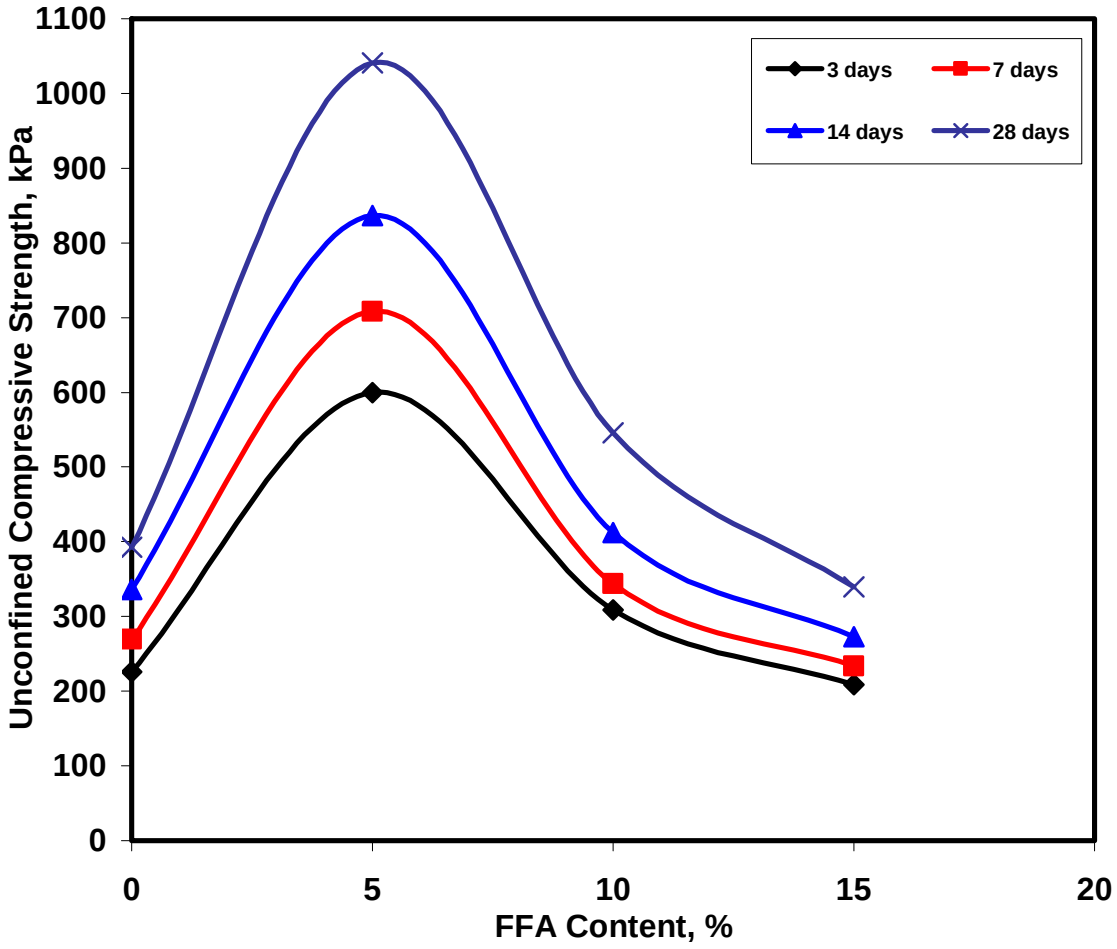

Figure 4-36: Variation of the q_u with FFA content for FFA-Non-plastic Marl Mixtures

Table 4-6: Unconfined Compressive Strength Test Results for FFA-Marl Soil

Cement (%)	FFA (%)	Unconfined Compressive Strength (kPa)			
		3 days	7 days	14 days	28 days
0	0	225.55	269.51	336.05	392.53
5	0	708.35	964.61	1203.29	1553.06
5	5	963.075	1386.26	1595.05	2151.415
5	10	882.745	1223.07	1427.355	1822.9
5	15	814.555	1140.71	1321.42	1625.93
0	5	599.555	708.355	836.475	1041.205
0	10	309.115	343.79	412.635	545.6
0	15	209.08	233.845	273.185	339.34

4.3.4 Durability (wetting and drying) Test

The ASTM D 559 standard durability test and the modified slake durability test were used on CKD-non-plastic marl and FFA-non-plastic marl mixtures with or without cement as listed in the Table 3.1. All CKD-marl soil and FFA-marl soil mixtures, except the 30% CKD one, collapsed during the first cycle and, therefore, they were considered as "failed" in the durability test.

Figure 4.37 presents the results obtained from the two durability tests conducted on the CKD-marl soil mixtures with 2% cement. It is seen that as the CKD content increased the weight loss decreased. The average weight losses of the all mixtures at the end of 12 cycles are shown in Table 4.7. The results indicate that the maximum weight loss for the all mixtures, except for 30% CKD, did not exceed the maximum allowable

weight loss of 14% as set forth by the Portland Cement Association (PCA) for cement-soil mixtures.

Similarly, all marl soil mixtures stabilized with FFA only collapsed during the first cycle and, therefore, considered as "failed" in the durability test. The weight loss of the FFA-marl soil mixtures with the addition of 5% cement is depicted in Figure 4.38. It is noticed that as the FFA content increased, the weight loss decreased.

The average weight losses of the all mixtures at the end of 12 cycles are summarized in Table 4.8. It is noticed that the average weight loss after 12 cycles for the all mixtures did not exceed the maximum allowable weight loss of 14% as set forth by PCA. It is worth mentioning that hairline cracks were observed in almost all CKD-cement-marl samples (2% cement with 5, 10, and 20% CKD) for durability test as shown in Figure 4.39. This is probably due to the cohesionless nature of the non-plastic marl [Al-Gunaiyan, 1998]. These cracks, however, were not deep and they seem to be very superficial.

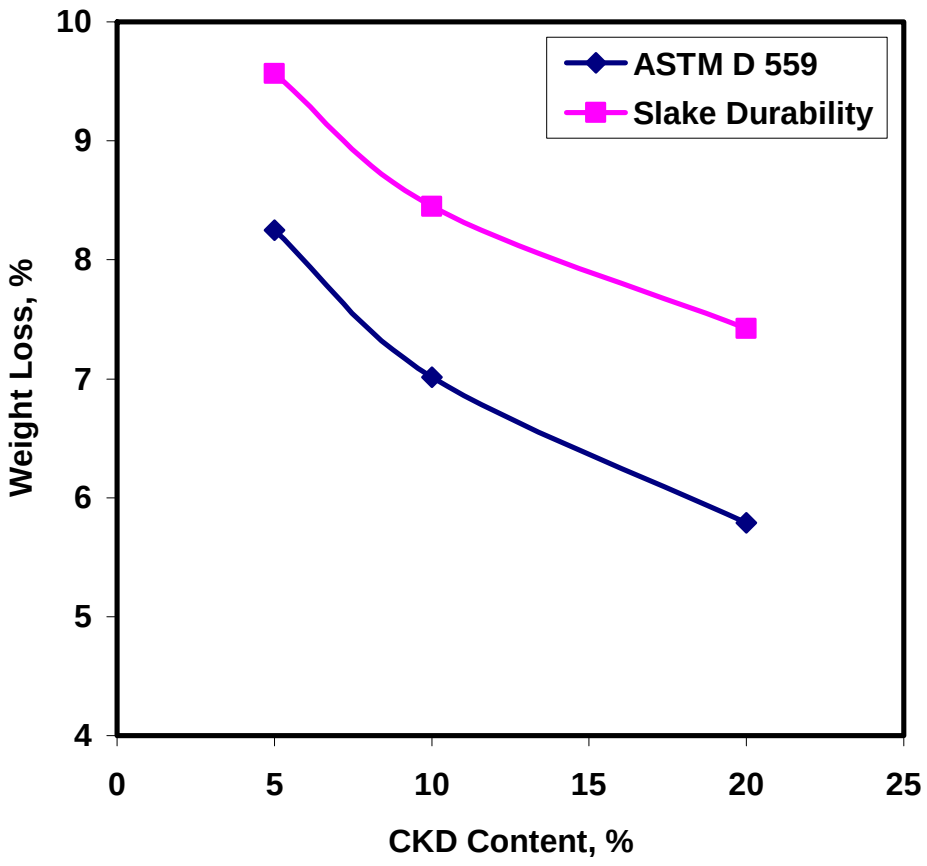

Figure 4-37: Variation of the Weight Loss with CKD Content and 2% Cement for Stabilized Non-Plastic Marl Soil

Table 4-7: Weight Loss for CKD-Marl Mixtures after 12 Cycles

CKD (%)	Cement (%)	Weight Loss (%)					
		ASTM D 559			Slake Durability		
		Sample # 1	Sample # 2	Average	Sample # 1	Sample # 2	Average
5	2	7.8	8.7	8.2	9.1	10.1	9.6
10	2	6.8	7.2	7.0	8.9	8.0	8.4
20	2	5.5	6.1	5.8	7.8	7.1	7.4
30	0	13.7	14.6	14.2	16	14.7	15.3

Table 4-8: Weight Loss of FFA-Marl after 12 Cycles

FFA (%)	Cement (%)	Weight Loss (%)					
		ASTM D 559			Slake Durability		
		Sample # 1	Sample # 2	Average	Sample # 1	Sample # 2	Average
5	5	9.2	9.5	9.4	10.7	9.3	10
10	5	7.2	8.2	7.7	8.7	8.0	8.4
15	5	7.1	6.7	6.9	7.4	8.3	7.8

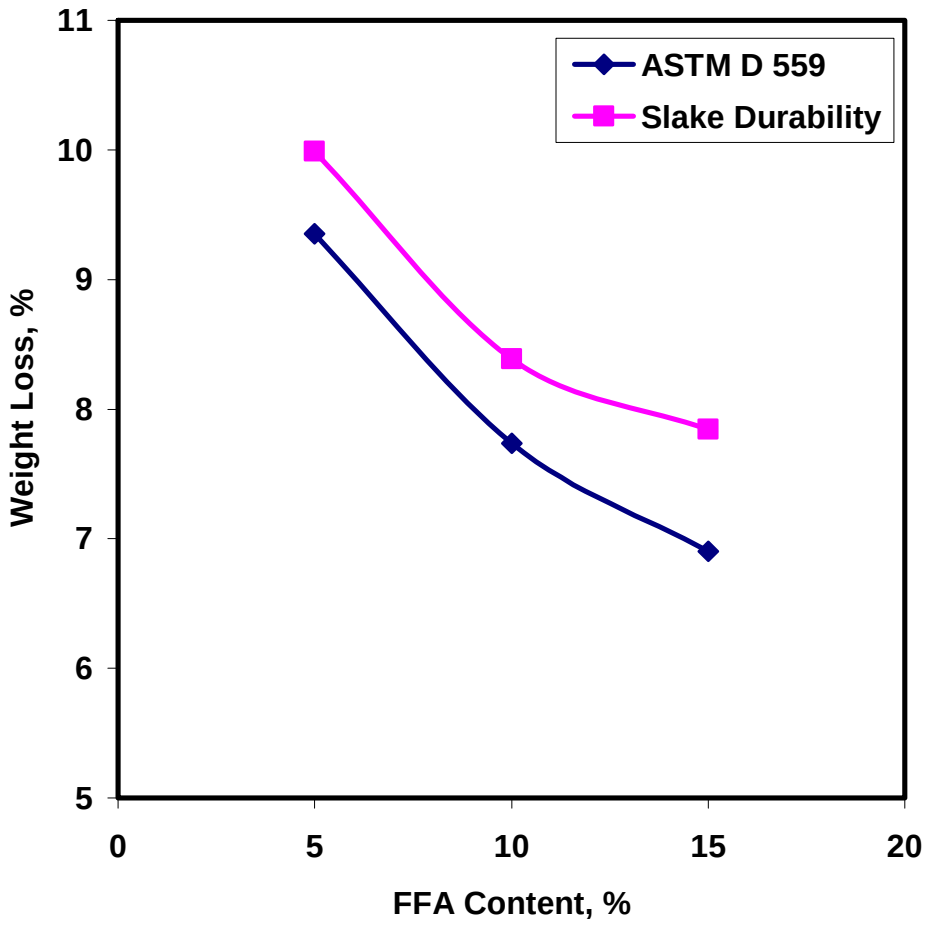

Figure 4-38: Variation of the Weight loss with FFA Content and 5% Cement for Stabilized Non-Plastic Marl

Figure 4-39: Durability CKD-Marl Samples with Hairline Cracks

4.4 Characterization of Sand Soil

According to the ASTM and AASHTO standards, tests were conducted to identify eastern Saudi sand with respect to its particle size. In this investigation, characterization tests included specific gravity of the solid grains and grain size distribution.

4.4.1 Specific Gravity Test Results

Two representative samples of sand were subjected to the specific gravity test and the values obtained were 2.663 and 2.661 with an average value of 2.662. The results were consistent since the variation from the average was minimal. The specific gravity falls within the range specified for sand soil [Al-Gunaiyan, 1998].

4.4.2 Grain-Size Distribution Test Results

The grain-size distribution curves for the sand soil are depicted in Figure 4.40. It can be seen that there is no large variation between grain size distributions for both the dry and washed sieving. This is ascribed to the fact that sand is made up of quartz which is not affected much by washing. Since sands are non-plastic in nature, the results revealed that the collected sand sample is classified as A-3 according to the AASHTO system and SP according to the USCS. The coefficients of uniformity (C_u) using the dry and washed sieving are 2.8 and 3, respectively. The corresponding values for the coefficient of curvatures (C_c) are 1.16 and 1.19.

4.5 Chemical Stabilization of Sand Soil

In this investigation, CKD and FFA additives were used as chemical stabilizers. The effect of these stabilizers on the sand soil was studied and the suitable one (i.e. the one producing high strength), was selected for detailed stabilization program.

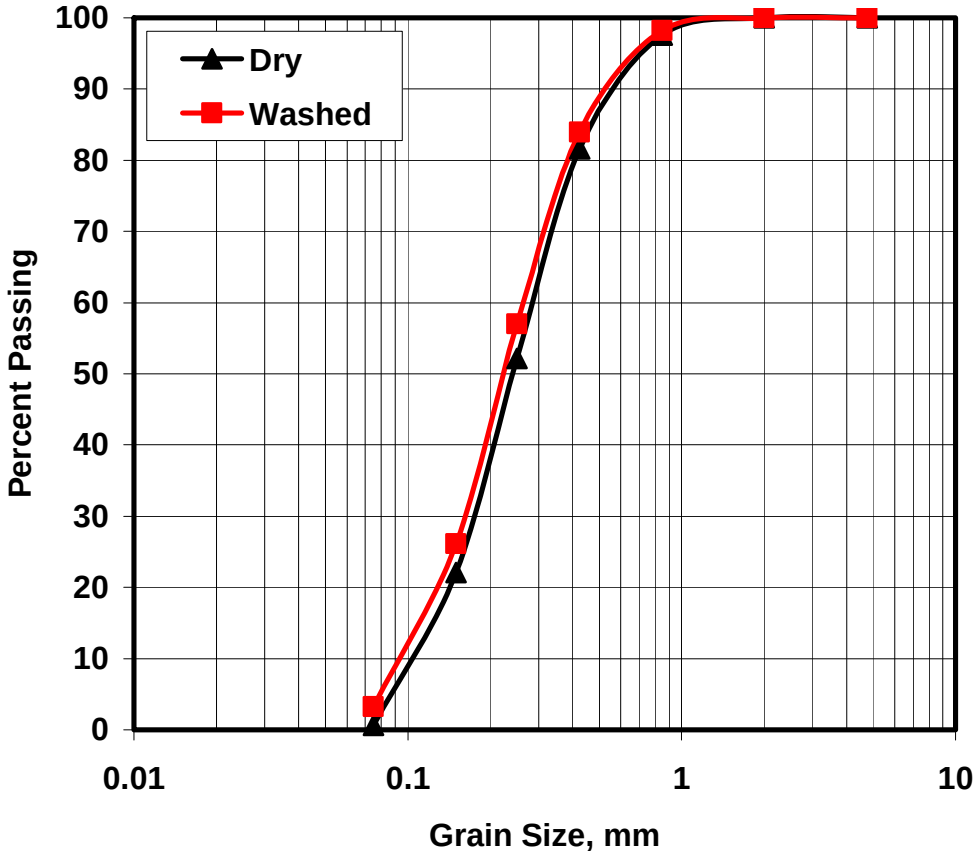

Figure 4-40: Grain-Size Distribution of Sand

4.5.1 Compaction Test Results of Sand

Modified Proctor compaction tests were conducted in order to establish the compaction characteristics of sand-CKD and sand-FFA mixtures. CKD and FFA additives were used in the range of 5 to 30%, as listed in Tables 3.2 and Table 3.3. The relationship between dry unit weight (γ_d) and water content for sand-CKD and sand-FFA mixtures were presented in Figures 4.41, 4.42, 4.43, and 4.44, respectively. A study of these figures indicates that there is an increase in the maximum dry unit weight $\{(\gamma_d)_{max}\}$ with the increase in CKD content with the usage of 2% cement or CKD alone. This is attributed to the fact that the CKD is very fine and it tends to fill up the voids between the sand particles, and thus the dry unit weight increases. Further, these figures reveal that the optimum moisture content tends to decrease with the increase in CKD content. The decrease in optimum moisture content is more pronounced for higher percentages of CKD. However, in the case of fuel fly ash (FFA) addition, the trend was reversed, whereby an increase in the FFA content resulted in a decrease in the maximum dry unit weight and an increase in the optimum moisture content. The reduction in the maximum dry unit weight is ascribed to the overdose of the FFA and lubricating effect. However, the increase in the optimum moisture content may be attributed to the fact that FFA is a very fine material, thus, any FFA addition needs more water for lubrication. It can be seen from Figure 4.44 that the change in moisture content is marginal when using FFA without cement.

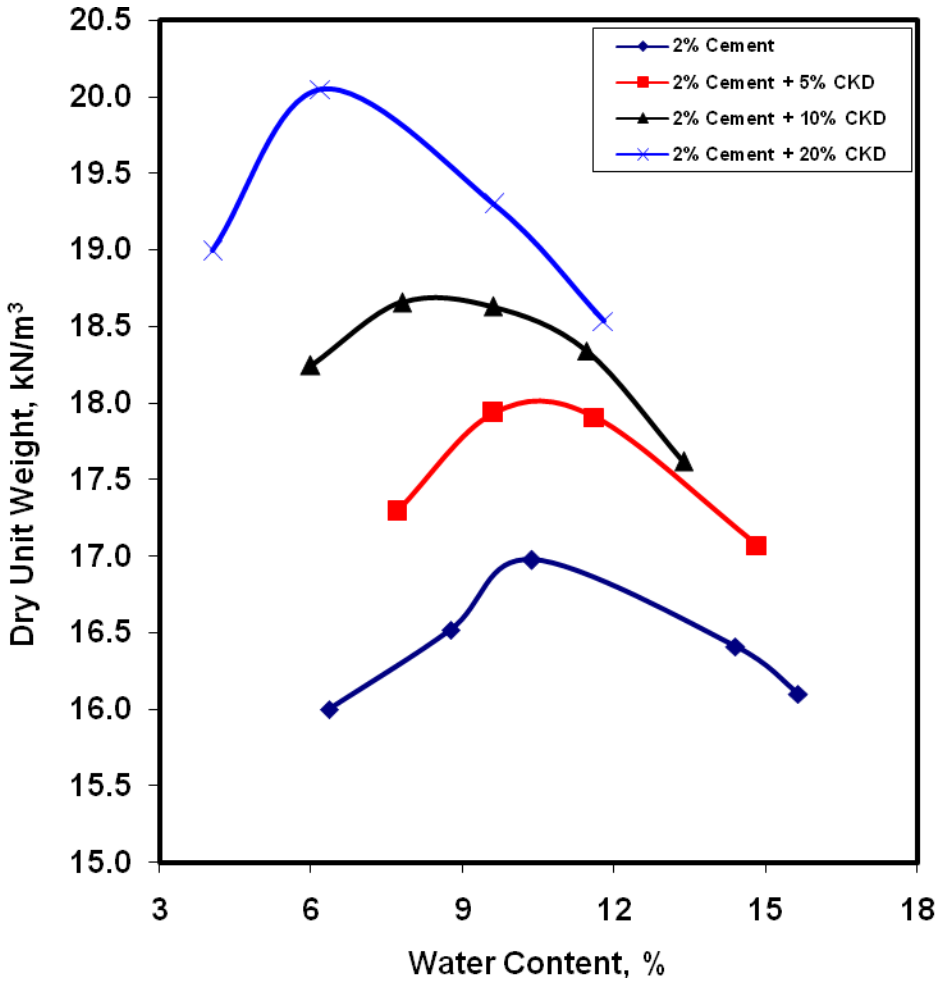

Figure 4-41: Effect of CKD Addition with 2% of Cement on Moisture-Unit Weight Relationship for Sand

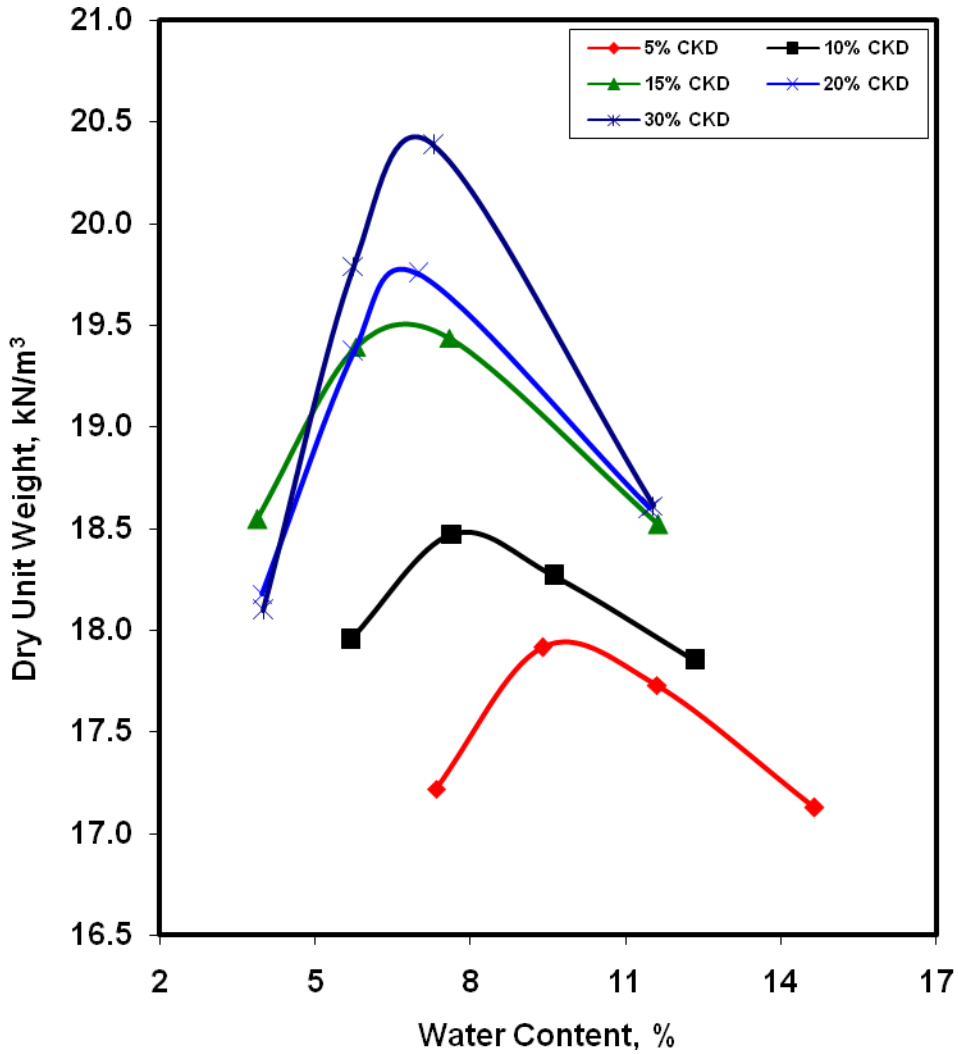

Figure 4-42: Effect of CKD Addition on Moisture-Unit Weight Relationship for Sand

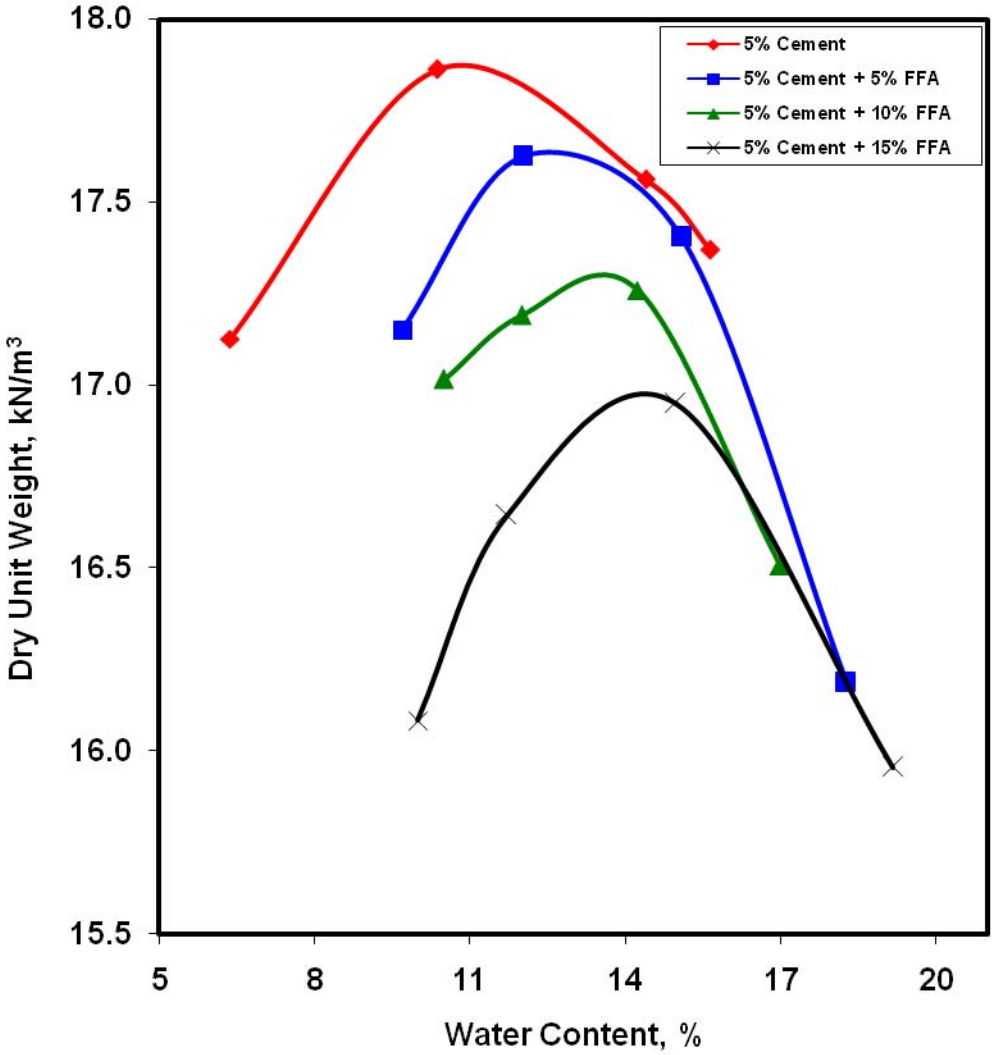

Figure 4-43: Effect of FFA Addition with 5% of Cement on Moisture-Unit Weight Relationship for Sand

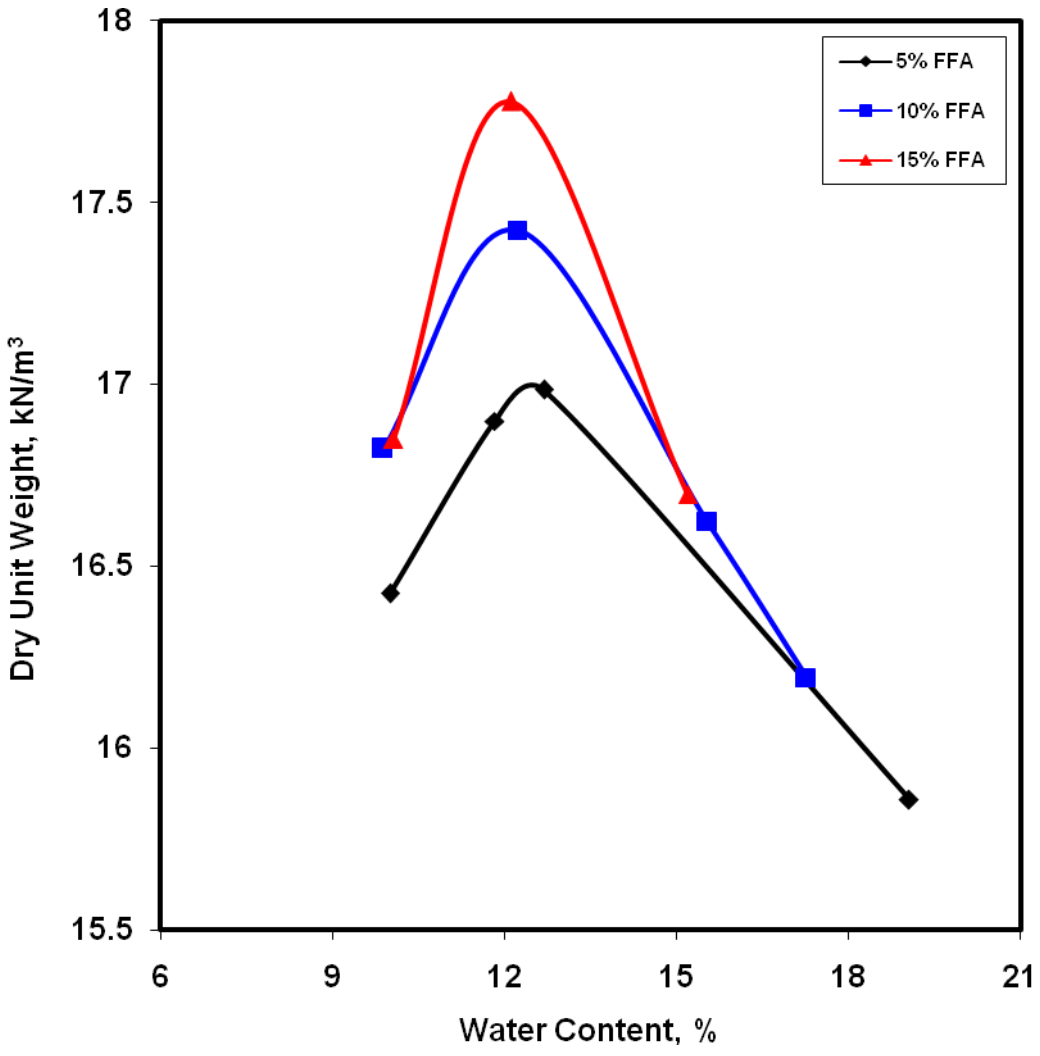

Figure 4-44: Effect of FFA Addition on Moisture-Unit Weight Relationship for Sand

4.5.2 CBR Test Results

All compaction specimens were subjected to CBR test and the moisture-density-CBR relationships are presented for all mixes. Figure 4.45 presents the moisture-unit weight-CBR relationship for sand soil treated with 2% cement as a reference. It is noticed that the maximum dry unit weight $\{\gamma_{d(max)}\}$ was 17 kN/m^3 at an optimum moisture content of 11.4%. From the same figure, it could be noted that the compaction curve followed the typical γ_d-w relationship whereby γ_d increases initially with the increase in moisture content until it reaches the maximum dry unit weight at the optimum moisture content (w_{opt}). Further increase in the moisture content resulted in a reduction in dry unit weight. Results showed that the maximum CBR was 181 at a moisture content of 9.8%. Similarly, it can be observed from Figure 4.45 that the CBR values increased with increasing the moisture content until the maximum CBR was attained at a moisture content of 8.8%. Thereafter, the increase in moisture content led to a significant reduction in the CBR values. The results indicated that the moisture content for maximum CBR is well below the optimum moisture content obtained from the dry unit weight-moisture content relationship. This is in agreement with the findings that reported by Al-Amoudi et al. (1992a) and Aiban et al. (1995).

Figure 4.46 depicts the moisture-unit weight-CBR relationship for sand soil treated with 2% cement and 5% CKD. The data in this figure indicate that the maximum dry unit weight was 18 kN/m3 at an optimum moisture content of 10.5%. However, the maximum value of the CBR was 232 at a moisture content of 9.6%. Comparison of the results of Figure 4.46 with those of Figure 4.45 indicates that there was an increase in the maximum dry unit weight and CBR values whereas the change in moisture content was

marginal due to the addition of 5% CKD to the stabilized sand with 2% cement. Further, the results indicate that there is a similarity between the CBR-moisture curve and dry unit weight-moisture curve.

Figure 4.47 shows the moisture-unit weight-CBR relationship for the stabilized sand with 2% cement and 10% CKD. The results indicate that the maximum dry unit weight was 18.68 kN/m^3 at an optimum moisture content of 8.7%. Further, the maximum CBR value of 351 was attained at a moisture content of 7.8%. The data in Figure 4.47 indicate that there was an increase in both the maximum dry unit weight and maximum CBR value when the 2% cement plus 10% CKD were added. Again, there is a similarity between the CBR-moisture curve and maximum dry unit weight-moisture curve. It is noticed that the maximum CBR value was attained at moisture content less than the optimum moisture content. When comparing the data in Figure 4.47 with that in Figure 4.46, it is observed that there was a little increase in the maximum dry unit weight and a significant increase in the CBR value after the addition of 10% CKD to the 2% cement in the mixture.

The relationship between moisture-unit weight-CBR for the stabilized sand with 2% cement and 20% CKD is presented in Figure 4.48. It is seen that the maximum dry unit weight was 20 kN/m^3 at an optimum moisture content of 6.2% while the maximum value of the CBR was 484. The maximum dry unit weight and maximum CBR value were attained at an optimum moisture content of 6.2%. Comparison the data of both curves indicates that there is an increase in both the maximum dry unit weight as well as the CBR value when 2% cement and 20% CKD were added.

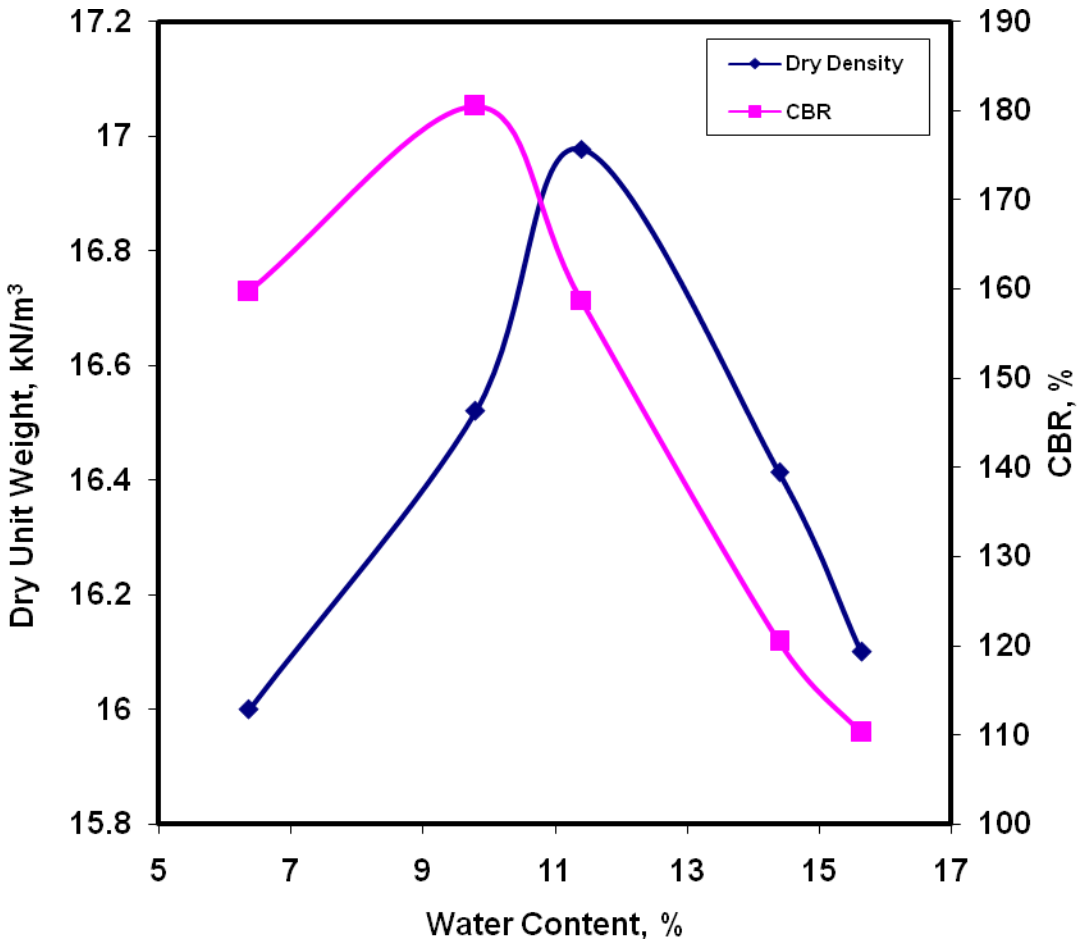

Figure 4-45: Moisture-Unit Weight-CBR Relationship for Sand Soil with 2% Cement Addition

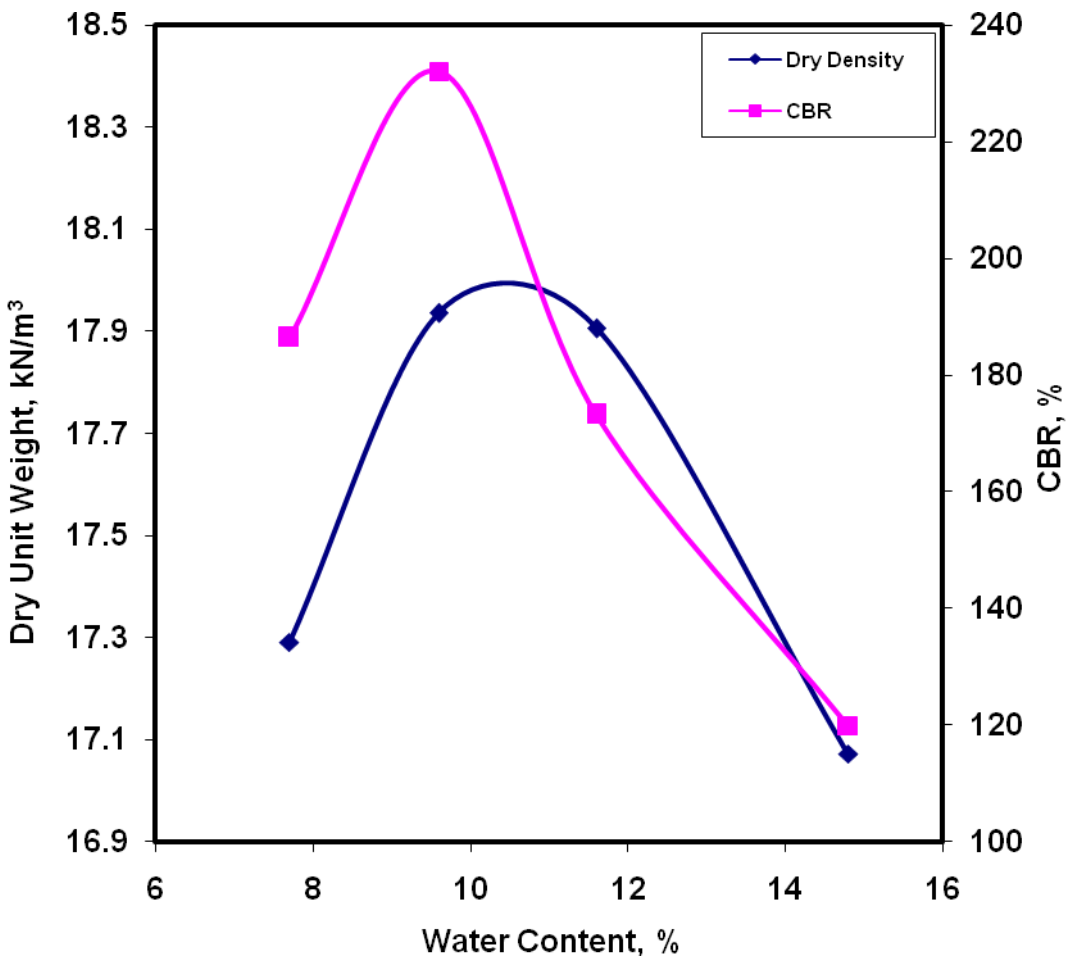

Figure 4-46: Moisture-Unit Weight-CBR Relationship for Sand Soil with 2% Cement and 5% CKD Additions

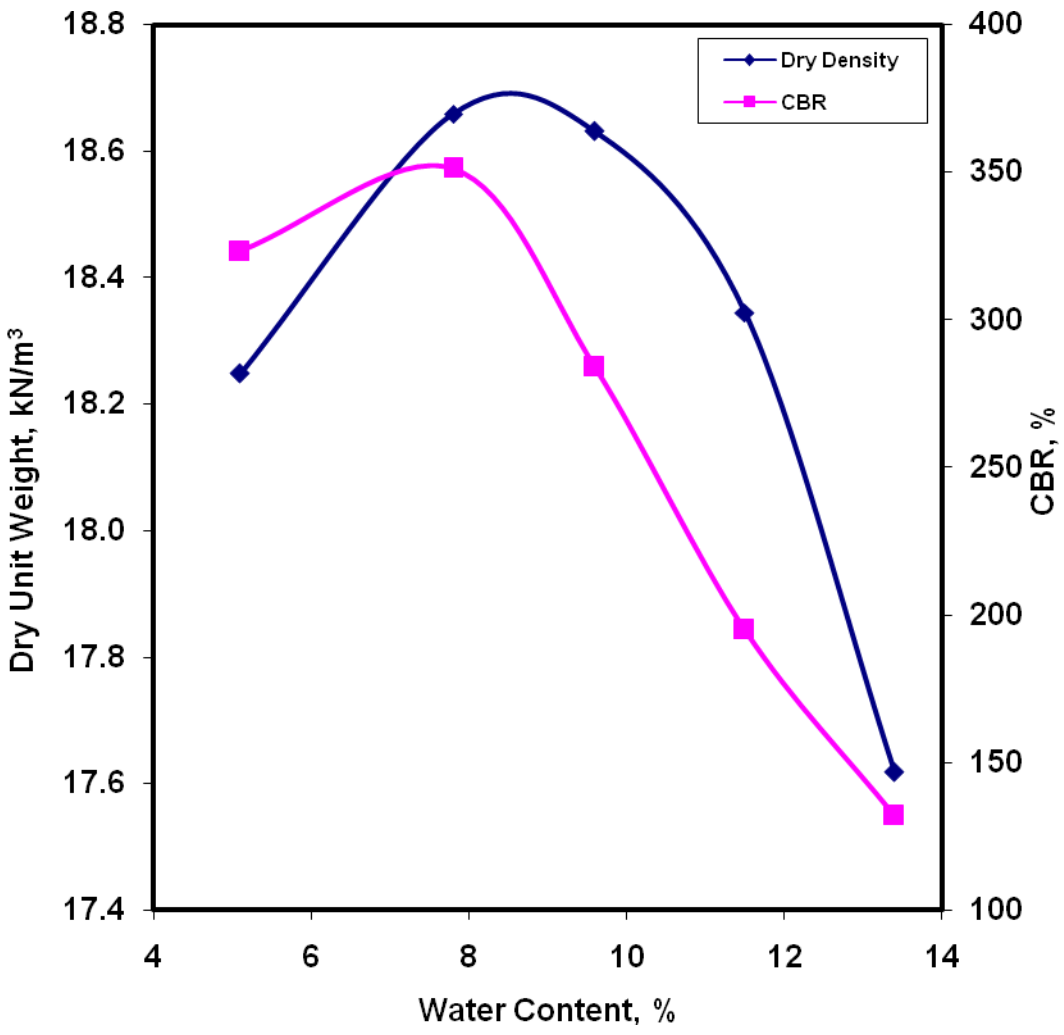

Figure 4-47: Moisture-Unit Weight-CBR Relationship for Sand Soil with 2% Cement and 10% CKD additions

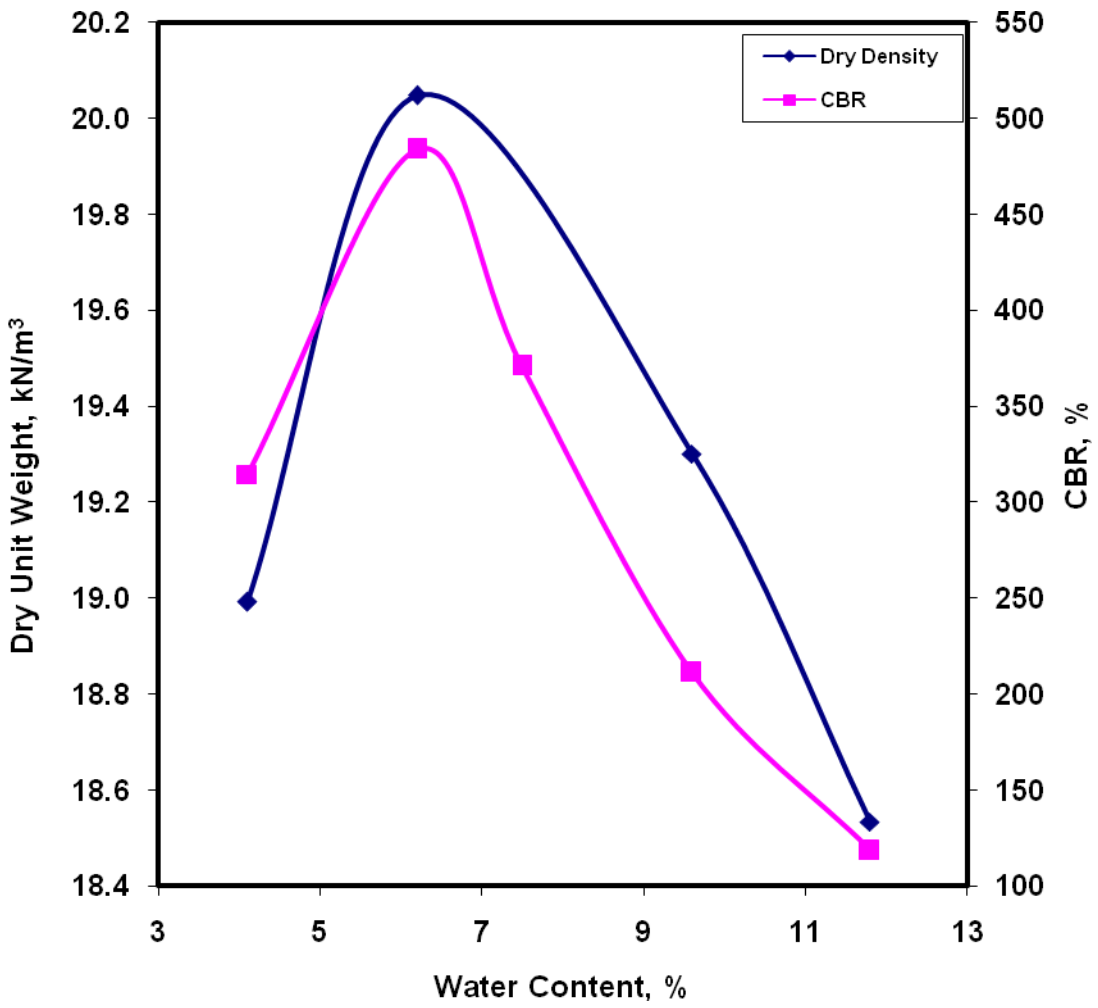

Figure 4-48: Moisture-Unit Weight-CBR Relationship for Sand Soil with 2% Cement and 20% CKD Additions

Figure 4.49 depicts the moisture-unit weight-CBR relationship for the stabilized sand with 5% CKD alone (0% cement). The results therein indicate that the maximum dry unit weight was 17.9 kN/m^3 at an optimum moisture content of 10%. On the other hand, the maximum CBR value was 78 at a moisture content of 8.3%. Similarity is seen between the CBR-moisture curve and dry unit weight-moisture curve. The CBR value increased with increasing the moisture content until the maximum value was attained. Thereafter, further increase in the moisture content leads to a decrease in the CBR value.

The relationship between the moisture-unit weight-CBR parameters for treated sand with 10% CKD is shown in Figure 4.50. It is seen from this figure that the maximum dry unit weight was 18.5 kN/m^3 at an optimum moisture content of 8%. Further, the maximum CBR value was 139 at a moisture content of 6.7%. Comparison of the data in Figure 4.50 with that in Figure 4.49 indicates that there was an increase in the maximum dry unit weight and significant increase in the CBR value. Moreover, the data indicate that there is a similarity between the CBR-moisture curve and the dry unit weight -moisture curve.

Figure 4.51 presents the moisture-unit weight-CBR relationship for stabilized sand with 15% CKD. From the data in the figure, the maximum dry unit weight was noted to be 19.5 kN/m^3 at an optimum moisture content of 7%. However, the maximum CBR value was 269 at a moisture content of 5.8%. The CBR value was attained at a moisture content less than the optimum moisture content as reported by Al-Amoudi et al. (1992a) and Aiban et al. (1995). When comparing the data in Figure 4.51 with that in Figure 4.50, it is noted that there was an increase in the maximum dry unit weight and CBR value after the addition of 15% CKD to the sand. The increase in the CBR value

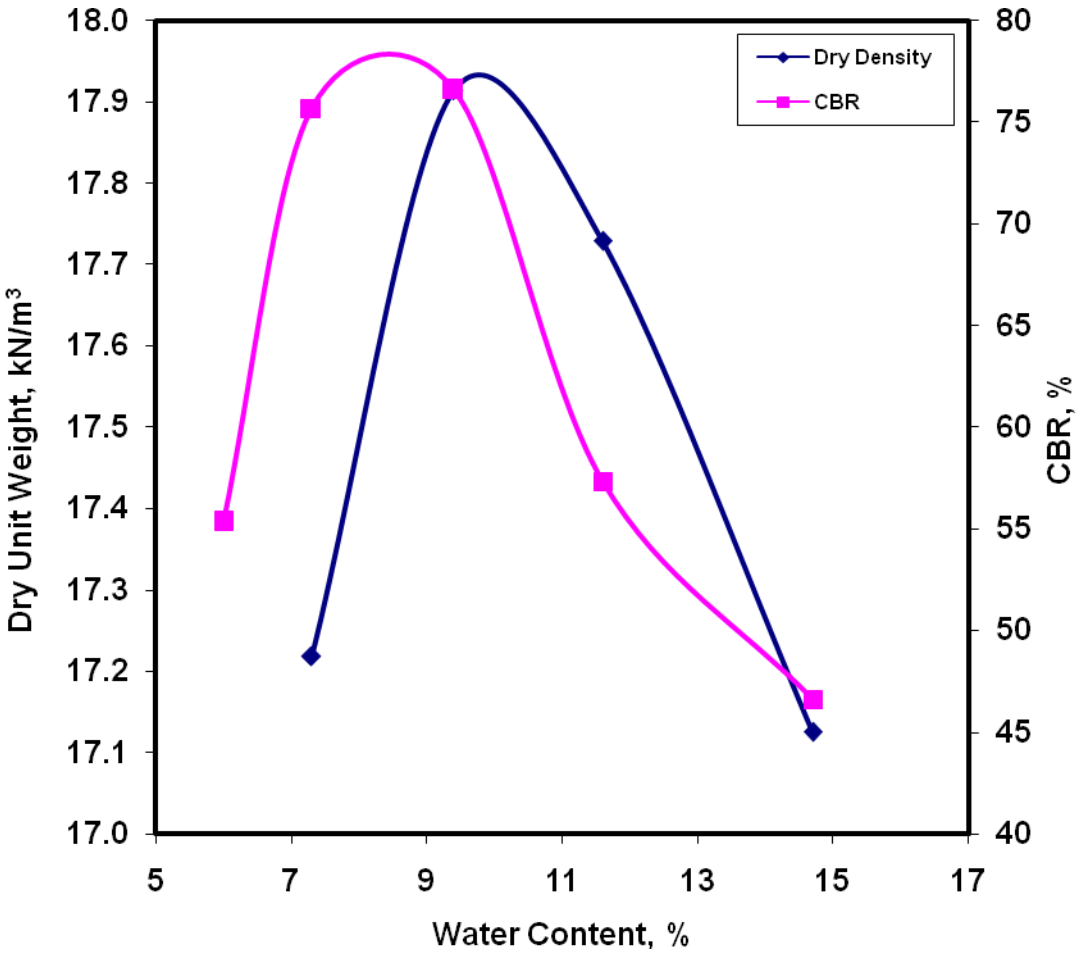

Figure 4-49: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% CKD Addition

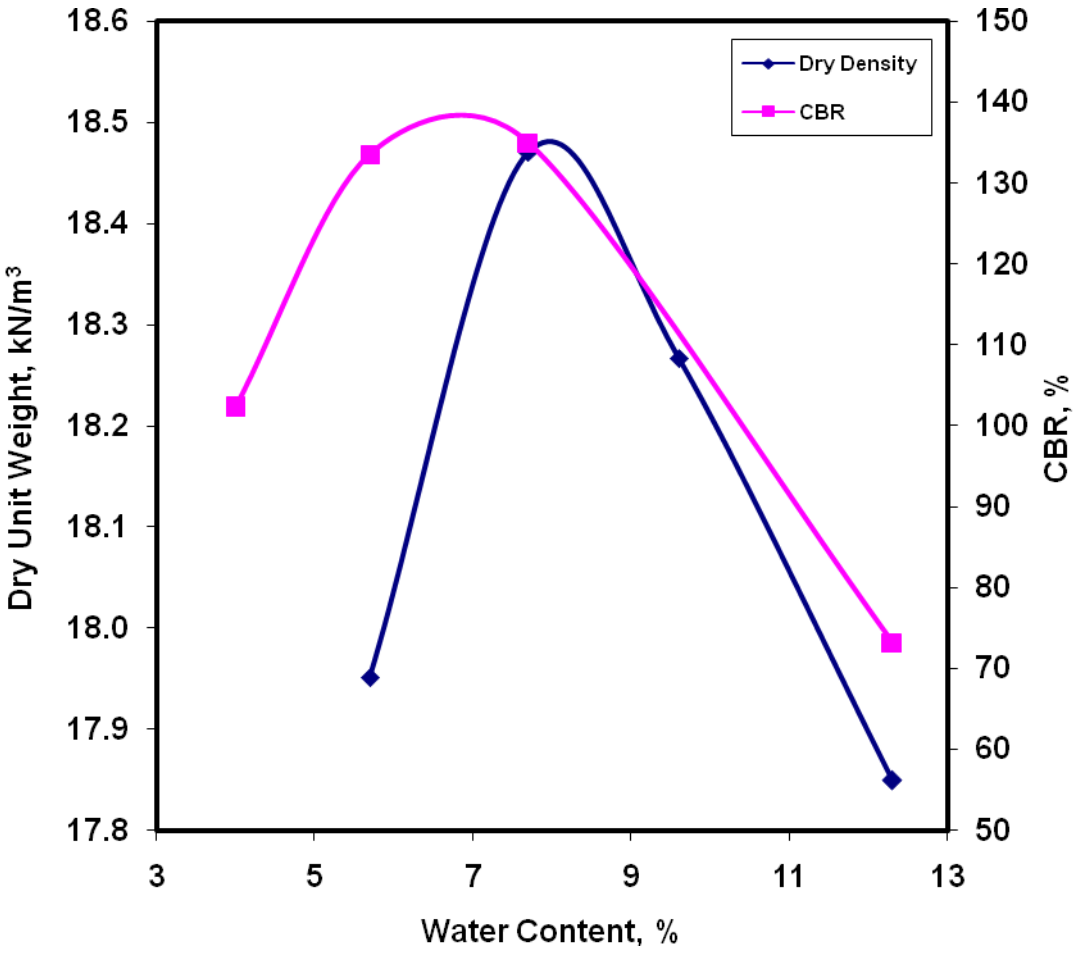

Figure 4-50: Moisture-Unit Weight-CBR Relationship for Sand Soil with 10% CKD Addition

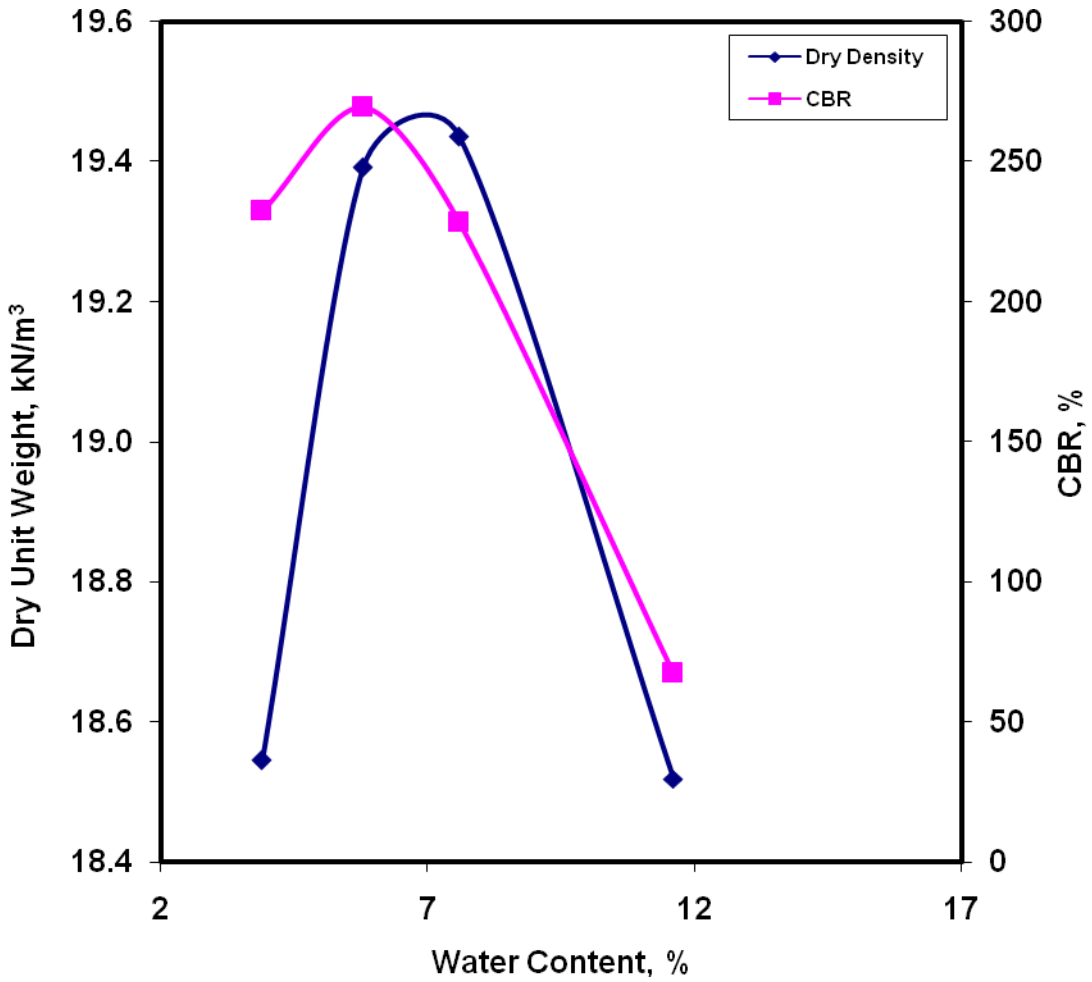

Figure 4-51: Moisture-Unit Weight-CBR Relationship for Sand Soil with 15% CKD Addition

was substantial. Furthermore, for the CBR curve, with the addition of 15% CKD, the CBR values increased initially with increasing the moisture content until it reached the maximum CBR value. Thereafter, further increase in the moisture content resulted in a sharp reduction in the CBR value reaching a value of 67 at a moisture content of 11.6%.

Figure 4.52 depicts the moisture-unit weight-CBR relationship for the stabilized sand with 20% CKD. It is seen from the data in this figure that the maximum dry unit weight was 19.8 kN/m^3 at an optimum moisture content of 7%. On the other hand, the maximum CBR value was 332 at a moisture content of 5.7%. The CBR-moisture curve has the same trend of the dry unit weight-moisture content curve. Comparison of the data in Figure 4.51 with that in Figure 4.52 indicates that there was a little increase in the maximum dry unit weight and significant increase in the maximum CBR value. However, the optimum moisture contents almost the same.

Figure 4.53 shows the moisture-unit weight-CBR relationship for the stabilized sand with 30% CKD. As shown in the Figure, the maximum dry unit weight was 20.4 kN/m^3 at an optimum moisture content of 7%. However, the maximum CBR value was 364.16 at a moisture content of 5.8%. Again, the CBR-moisture curve follows the same trend of the dry unit weight-moisture curve. Comparison of the data in both curves indicates that there is a high increase in the maximum dry unit weight as well as in the CBR value when 30% CKD additive was added to the sand.

Figure 4.54, Figure 4.55 and Table 4.9 summarize the CBR test results for sand stabilized with 2% cement and various dosages of CKD and with various percentages of CKD alone respectively. It can be seen that the CBR value increased from 181 for sand stabilized with 2% cement only to 232, 351, and 484 when 5, 10, and 20% of CKD were

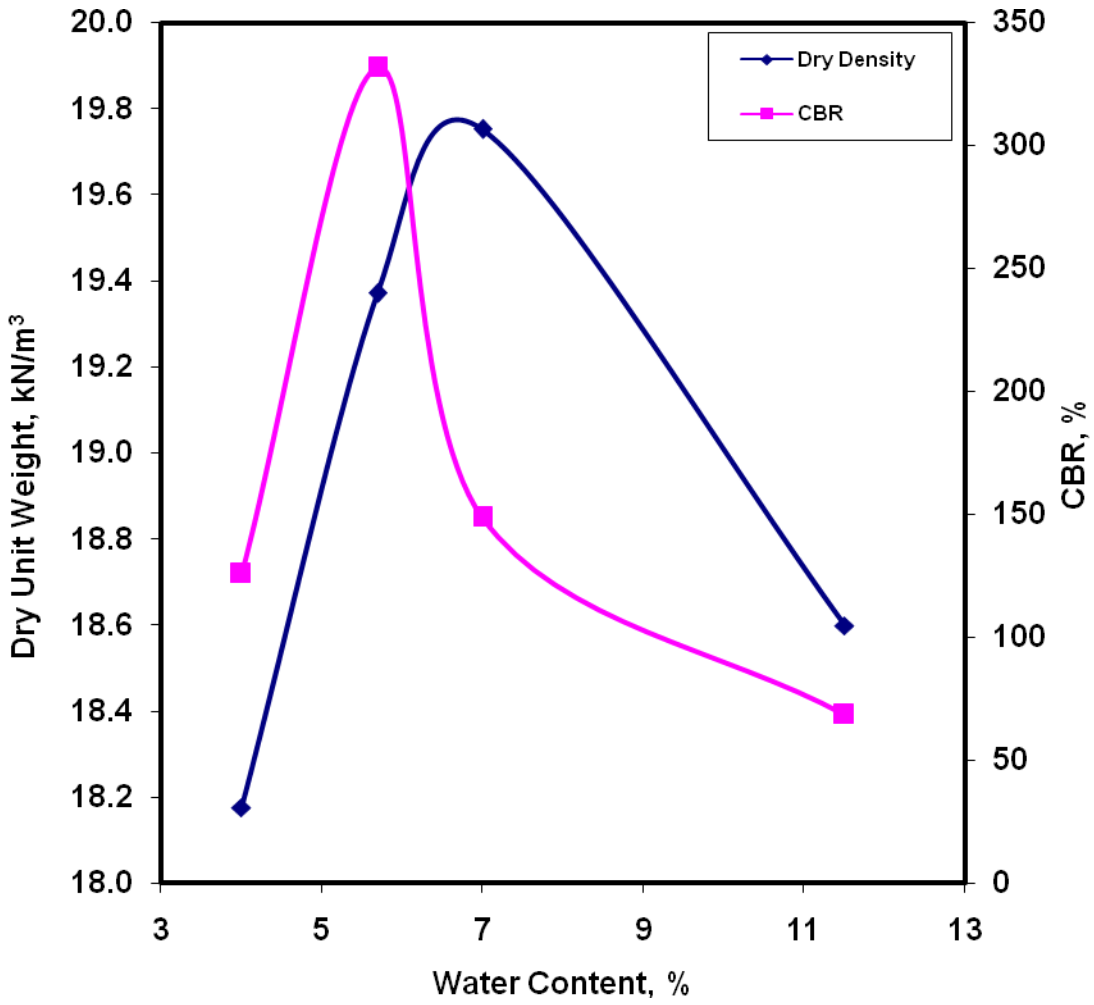

Figure 4-52: Moisture-Unit Weight-CBR Relationship for Sand Soil with 20% CKD Addition

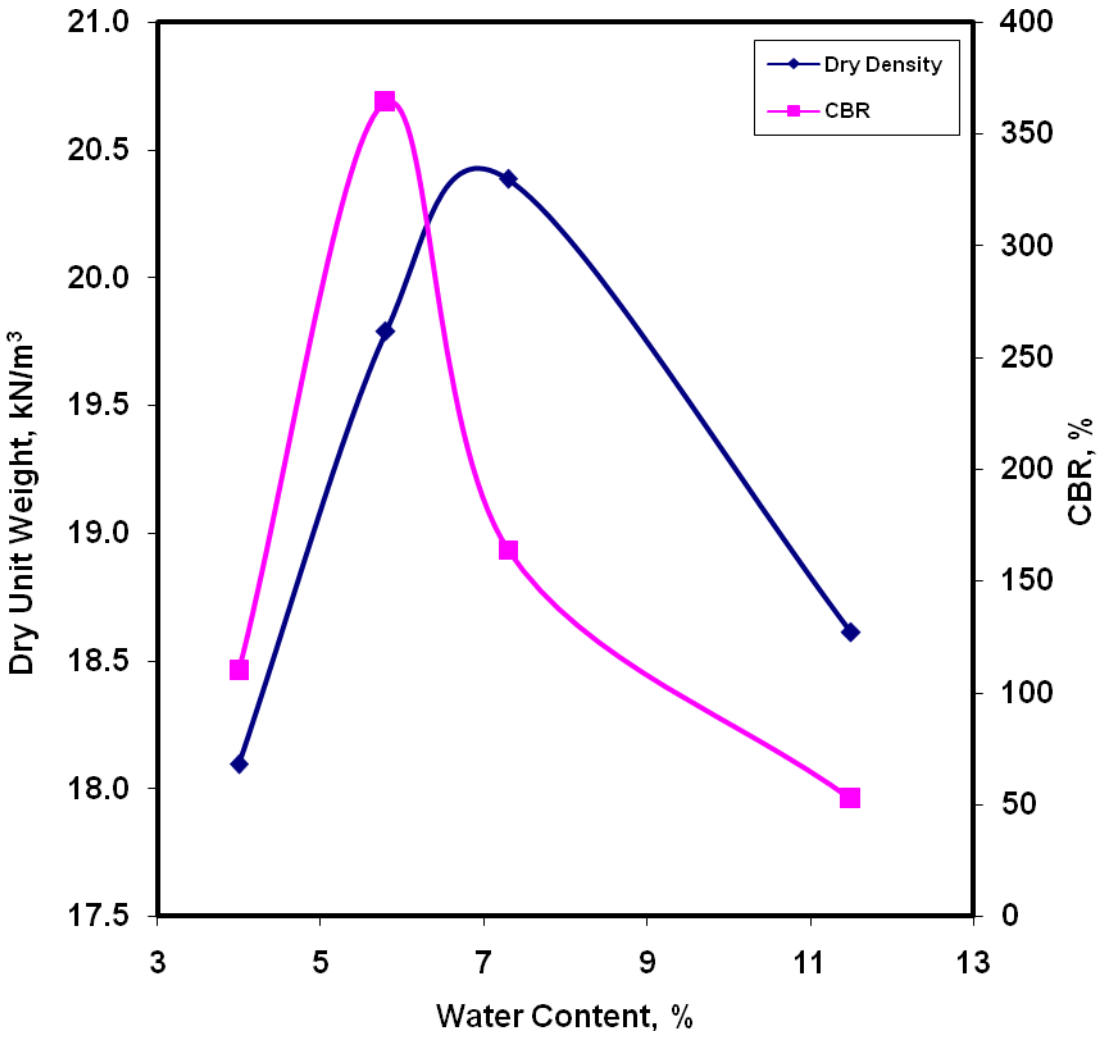

Figure 4-53: Moisture-Unit Weight-CBR Relationship for Sand Soil with 30% CKD Addition

added to the sand soil with 2% cement by weight of dry soil, respectively. Such obvious increase in the CBR values corresponds to an improvement ratio of 1.3, 1.9, and 2.7 times that of the stabilized sand with 2% cement. This increase in the CBR value due to the addition of CKD is much higher than the range of CBR of untreated sand. Similarly, the CBR values increased from 78 for sand stabilized with 5% CKD to 139, 270, 331, and 364 when 10, 15, 20, and 30% of CKD by weight of dry soil, respectively, were added. This substantial increase in CBR for 15, 20, and 30%CKD additions is much higher than that of the treated sand with 2% cement only. These data indicate that as the CKD content increases, CBR value increases. However, there was a very marginal decrease in optimum moisture content when the CKD content increased.

Figure 4.56 shows the relationship between the maximum CBR value and CKD content for the sand soil. It is noted that as the CKD content increases the maximum CBR value increases significantly. It is also seen that higher CBR values were attained when 2% cement was added to the CKD content. The sharp increase in the CBR with the increase in CKD is ascribed to the cementitious properties of the CKD.

Figure 4.57 presents the moisture content-unit weight-CBR relationship for the stabilized sand with 5% cement. The data reveal that the maximum dry unit weight was 17.9 kN/m^3 at an optimum moisture content of 11% while the maximum CBR value was 273 at a moisture content of 8.8%. Comparison of the data in Figure 4.57 with that in Figure 4.45, for the treated sand with 2% cement, indicates that there was an increase in the CBR value from 181 to 273 corresponds to an improvement ratio of 1.5 times.

Figure 4.58 shows the relationship of the moisture-unit weight-CBR for the stabilized sand with 5% cement and 5% FFA. The maximum dry unit weight was 17.6

kN/m^3 at an optimum moisture content of about 12%. On the other hand, the maximum CBR value was 120 at the same moisture content. Comparison the curves of Figure 4.58 and the curves of Figure 4.57 indicates that addition of FFA reduced the positive effect of cement substantially. The maximum CBR value decreased from 273 to 120 when 5% FFA was added. Similarly, the maximum dry unit weight decreased from 17.9 kN/m^3 to 17.6 kN/m^3. However, the optimum moisture content increased from about 11% to about 12%.

Table 4-9: Compaction and CBR Test Results for CKD-Sand Soil

Cement (%)	CKD (%)	$(\gamma_d)_{max}$ kN/m^3	w_{opt} (%)	CBR (%)	w (%)
2	0	16.978	11.4	181	9.8
2	5	18	10.5	232	9.6
2	10	18.68	8.7	351	7.8
2	20	20.048	6.2	484	6.2
0	5	17.93	10	78	8.3
0	10	18.481	8	139	6.7
0	15	19.5	7	269	5.8
0	20	19.775	7	332	5.7
0	30	20.4	7	364	5.8

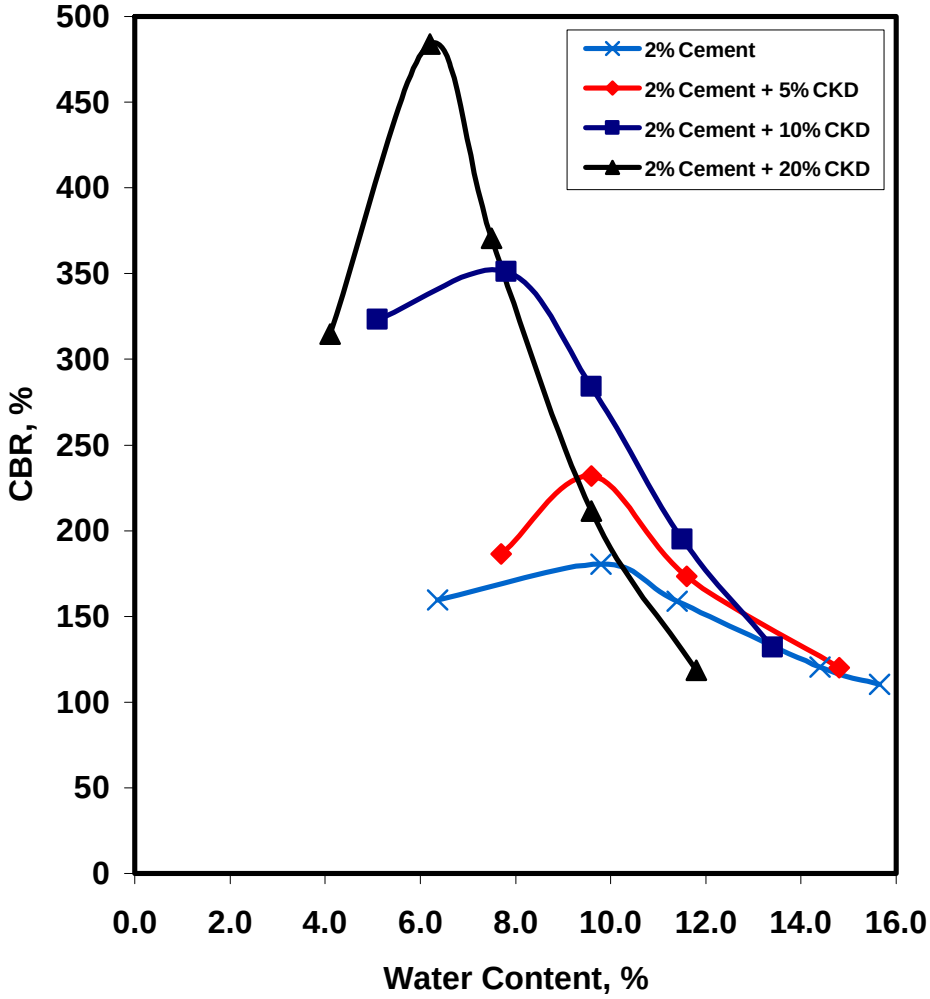

Figure 4-54: Effects of Moisture and 2% Cement with CKD Contents on CBR of Sand

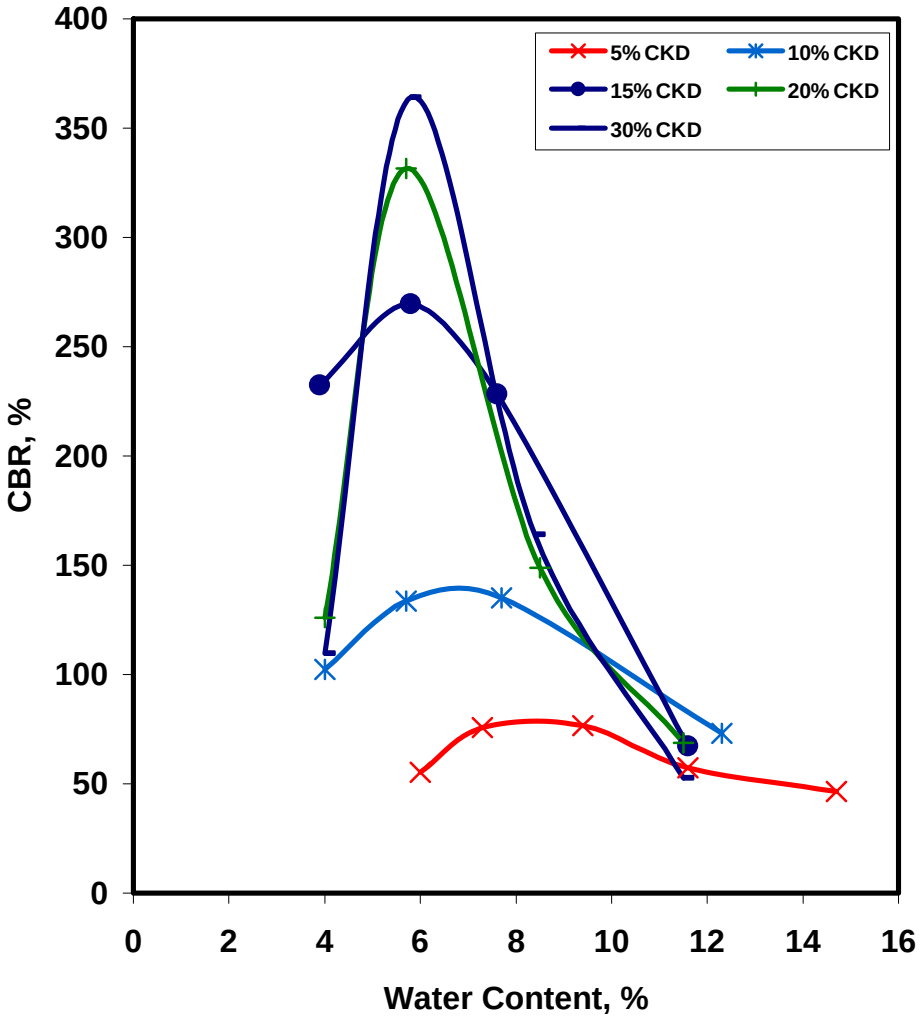

Figure 4-55: Effects of Moisture and CKD Contents on CBR of Sand

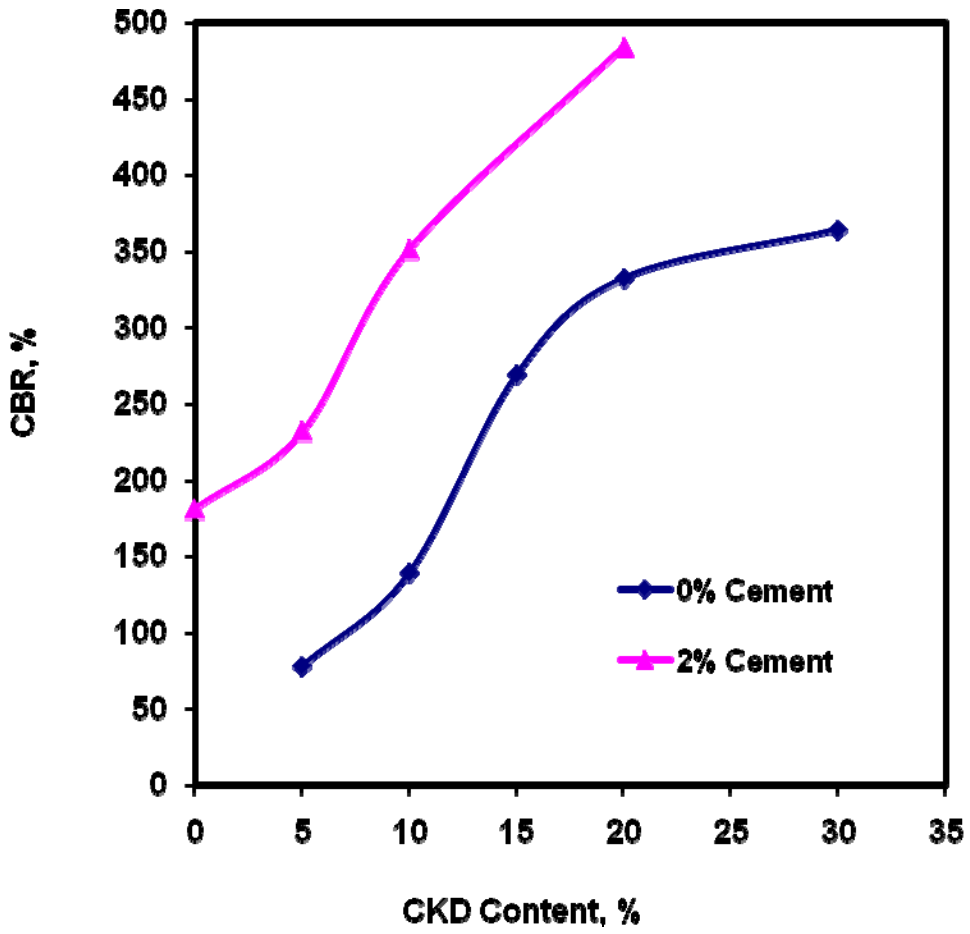

Figure 4-56: Maximum CBR Value-CKD Content Relationship for Sand Soil

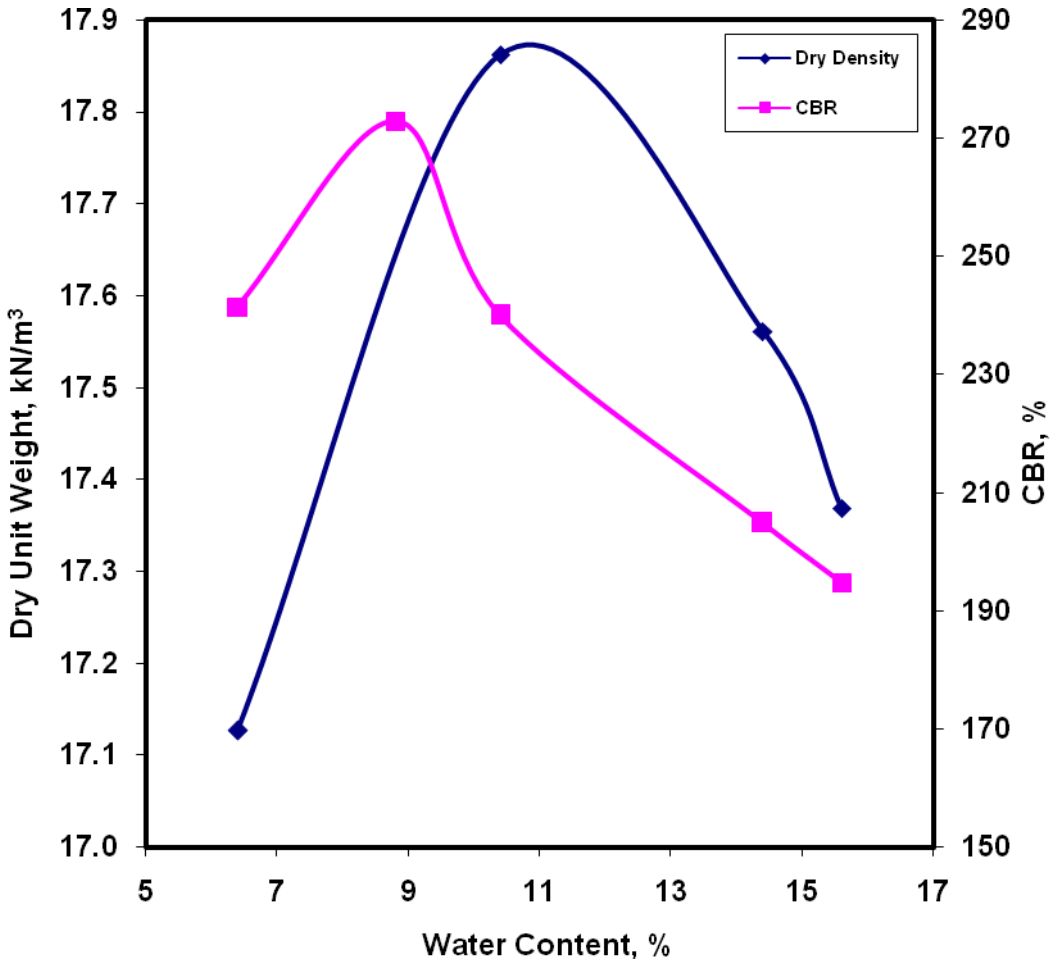

Figure 4-57: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% Cement Addition

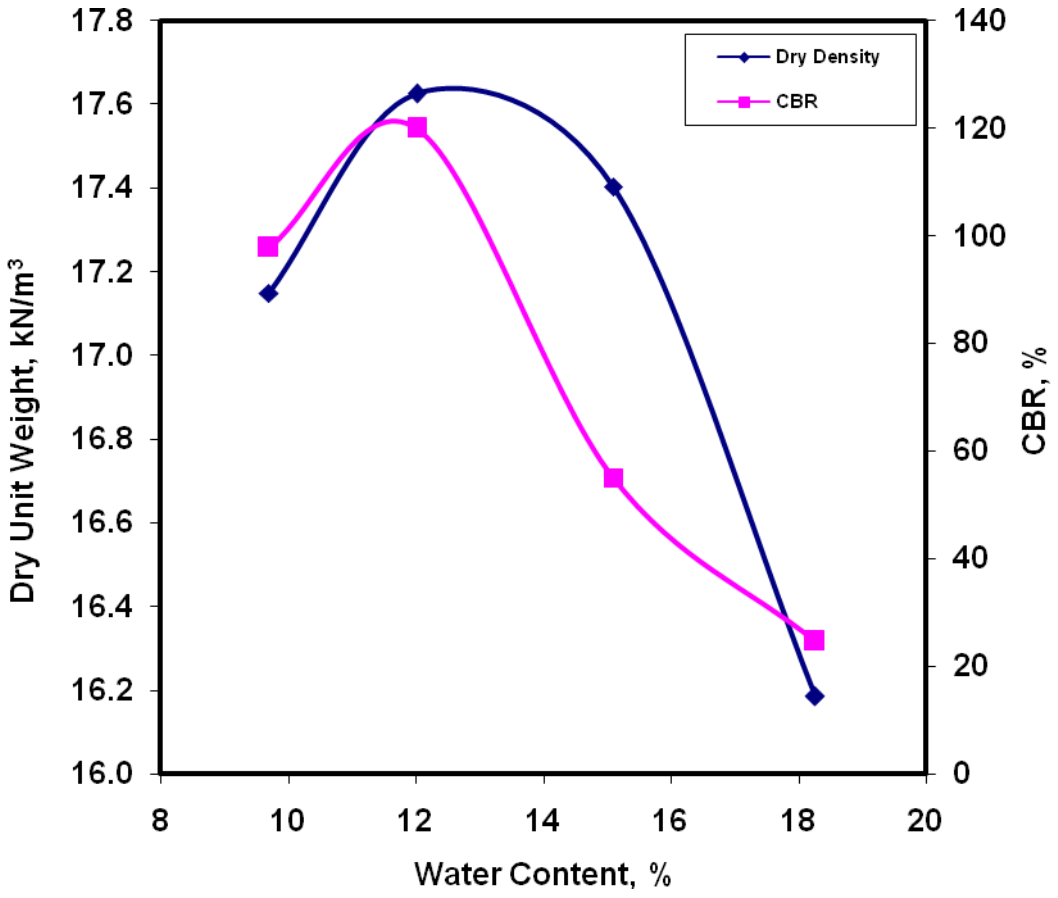

Figure 4-58: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% Cement and 5% FFA Additions

The relationship of the moisture-unit weight-CBR for the stabilized sand with 5% cement and 10% FFA is depicted in Figure 4.59. It can be seen that the maximum dry unit weight was 17.3 kN/m^3 at an optimum moisture content of 13.7% while the maximum CBR value was 133 at a moisture content of about 11.8%. Comparison of the data in Figure 4.59 and that in the Figure 4.58 shows that there was a little increase in the CBR value from 120 to 130 and a slight decrease in the dry unit weight value from 17.6 kN/m^3 to 17.3 kN/m^3 when 5% cement and 10% FFA were added to the sand soil. Although there was an increase in the CBR value, it was still much lower than that due to the addition of 5% cement additive which was 273 as shown in Figure 4.57.

Figure 4.60 depicts the relationship of the moisture-unit weight-CBR for the stabilized sand with 5% cement and 15% FFA. The data in the figure show that the maximum dry unit weight was 17 kN/m^3 at an optimum content of 14.3%. Similarly, the maximum CBR value was about 151 at a moisture content of about 11.7%. The maximum CBR value was attained at a moisture content less than the optimum moisture content. Comparison of the data in Figure 4.60 and the data in Figure 4.59 indicates that there was an increase in the CBR value when 5% cement and 15% FFA were added to the sand. Furthermore, there was an increase in the moisture content and a decrease in the maximum dry unit weight with increasing the ash content. Again, the maximum CBR value was much lower than that of the treated sand with 5% cement alone.

Figure 4.61 presents the relationship of the moisture-unit weight-CBR for the stabilized sand with 5% FFA alone (0% cement). It is observed that the maximum dry unit weight was 17 kN/m^3 at an optimum moisture content of 12.7%. On the other hand, the maximum CBR value was very low at about 36 at a moisture content of 11.8%. From

the same figure, it is noted that the CBR value was very low compared with that of 5% cement addition in Figure 4.57 which was 273.

Figure 4.62 shows the relationship of the moisture-unit weight-CBR for the stabilized sand with 10% FFA. The results from γ_d-w curve indicate that the maximum dry unit weight was 17.4 kN/m^3 at an optimum moisture content of 12.2%. For the CBR-moisture curve, however, the maximum CBR value was 40 at a moisture content of 12 %. Comparison of the data in Figure 4.62 with that in Figure 4.61 reflects that there was a marginal increase in the maximum dry unit weight from 17 kN/m^3 to 17.4 kN/m^3. Similarly, the CBR value increased from 36 to 40 due to the addition of 10% of FFA.

The relationship of the moisture-unit weight-CBR for the stabilized sand with 15% FFA was presented in Figure 4.63. It can be deduced that the maximum dry unit weight was 17.8 kN/m^3 at an optimum moisture content of 12.1%. On the other hand, for the CBR-moisture curve, the maximum CBR value was 68. The maximum dry unit weight and the maximum CBR value were attained at the same moisture content. Comparison of the data in Figure 4.63 with that in Figure 4.62 indicates that there was a similarity in the shape of the CBR-w and γ_d-w curves. In addition, there was an increase in the maximum dry unit weight $\{\gamma_{d(max)}\}$as well as in the maximum CBR value when 15% FFA was added. Again, comparison of the data in this figure with those in Figure 4.57 reveals that the maximum CBR value was still much lower than that of the treated sand with 5% cement which was 273.

Figure 4.64, Figure 4.65 and Table 4.10 summarize the CBR test results for FFA stabilized- sand mixtures. It is clear that as the FFA increases, the CBR increases and the optimum moisture content increases. However, all the ash-based samples had much lower

CBR values as compared with the 5% cement-stabilized sand samples. The data therein indicate that the CBR value decreased from 273 for the sand treated with 5% cement to 120 for the sand treated with 5% cement and 5% FFA and then increased to 133 and 151 when 10 and 15 % FFA with 5% cement additions were added. This indicates that the addition of FFA reduced the positive effect of cement substantially. The increase in the CBR value when FFA alone was added to the sand was negligible. The CBR value increased from 36 for 5% FFA addition to 40 and 68 for 10 and 15% FFA additions, respectively. Hence, it can be concluded that the FFA addition does not work with sands and, therefore, it is not beneficial in sand stabilization.

The maximum CBR value for the sand soil versus FFA content is presented graphically in Figure 4.66. It is observed that the addition of FFA dropped the CBR value sharply which means that FFA additive reduced the positive effect of the cement substantially. Although, there was an increase in the CBR value with the increase in FFA content, the maximum CBR value was still lower than that of the sand stabilized with 5% cement alone.

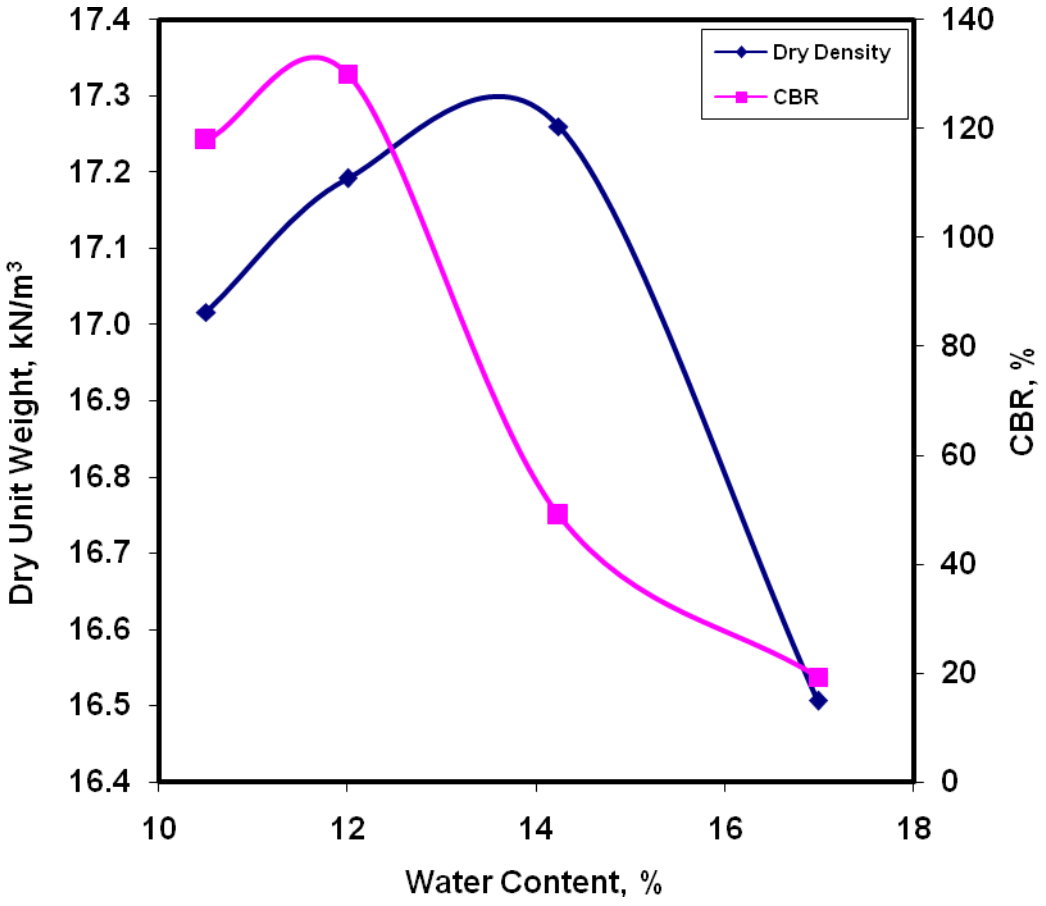

Figure 4-59: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% Cement and 10% FFA Additions

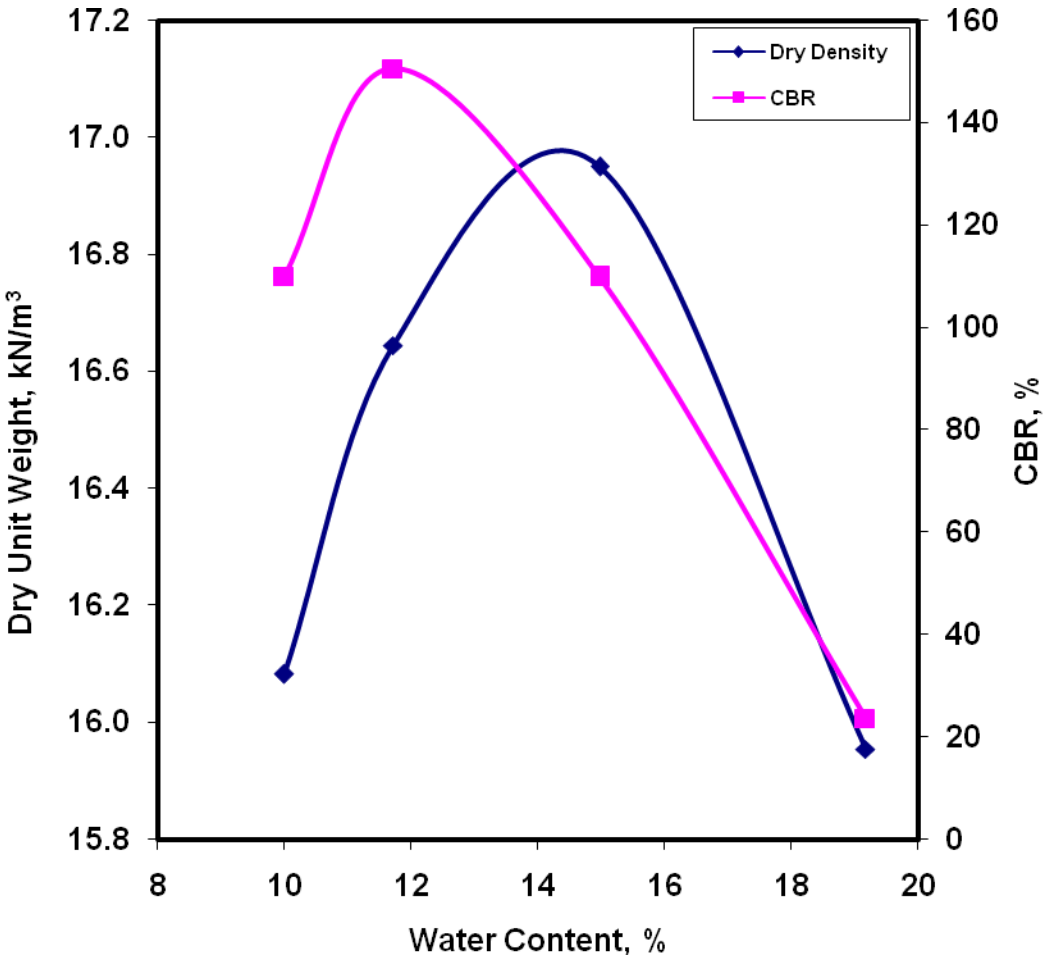

Figure 4-60: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% Cement and 15% FFA Additions

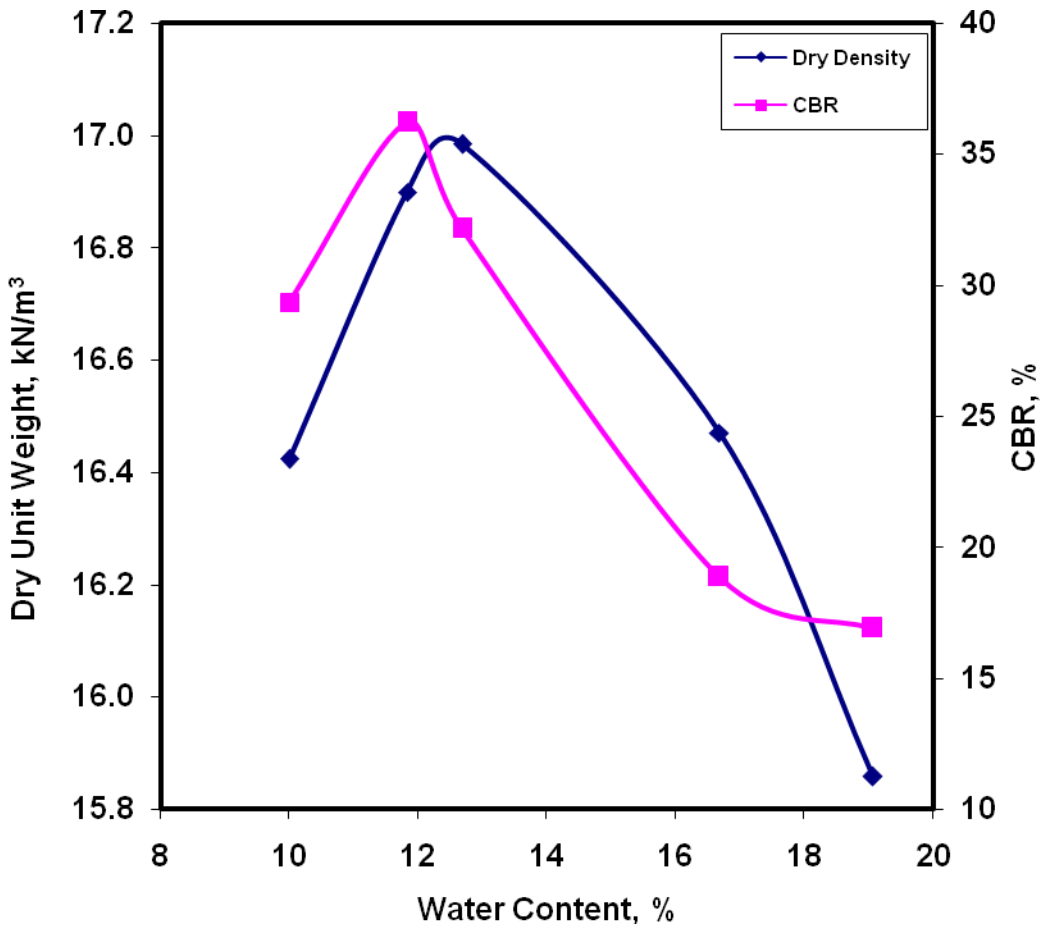

Figure 4-61: Moisture-Unit Weight-CBR Relationship for Sand Soil with 5% FFA Addition

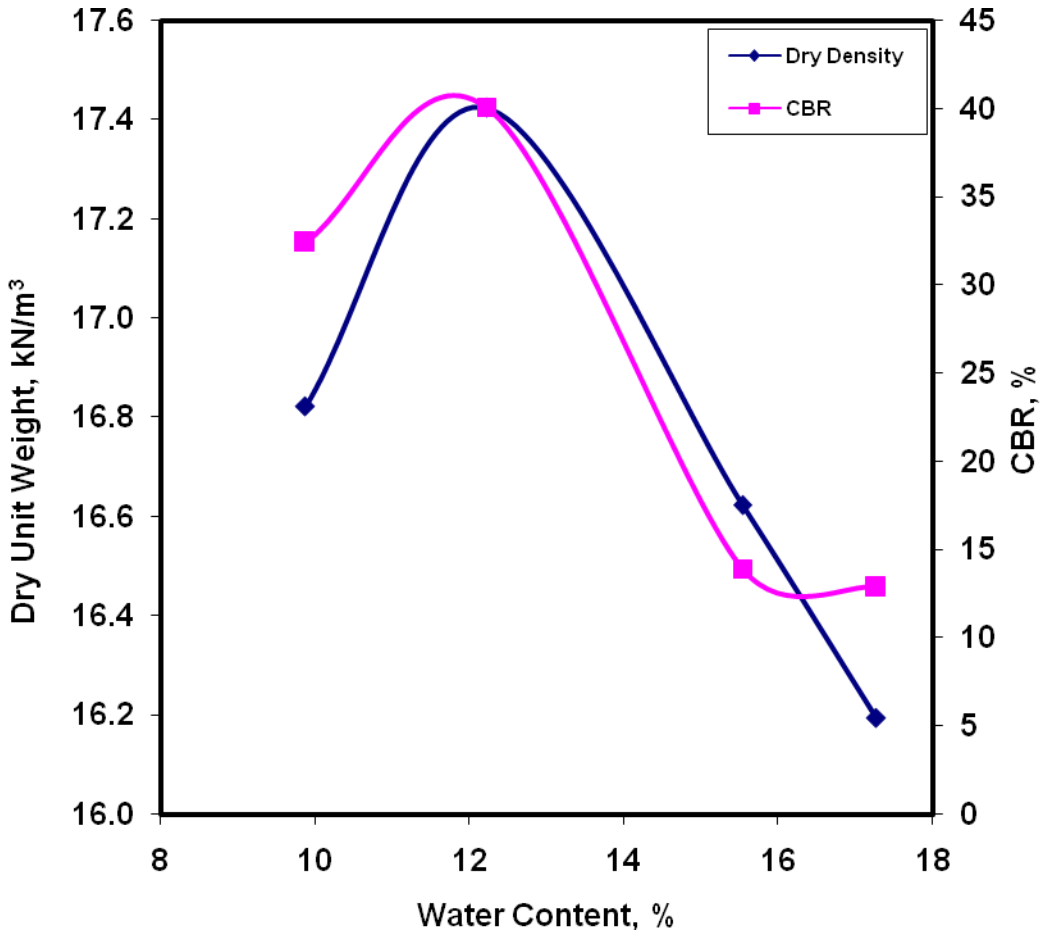

Figure 4-62: Moisture-Unit Weight-CBR Relationship for Sand Soil with 10% FFA Addition

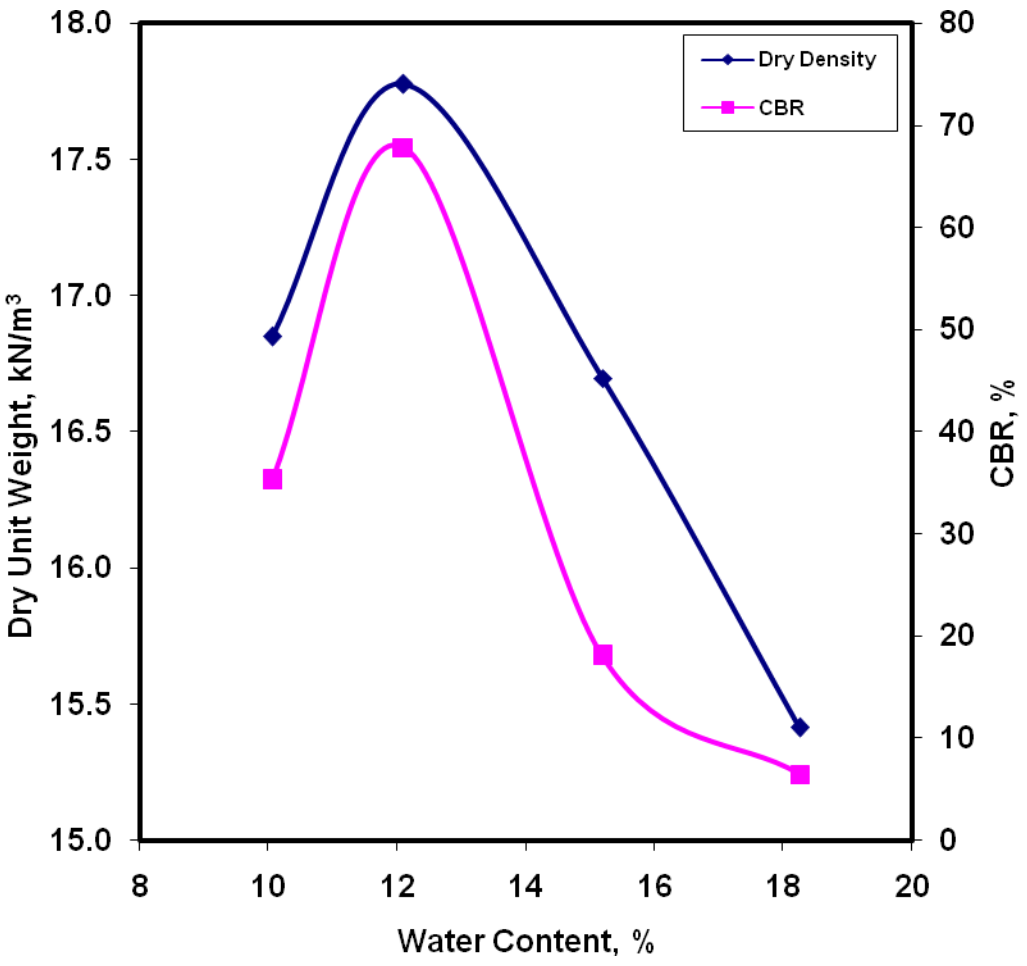

Figure 4-63: Moisture-Unit Weight-CBR Relationship for Sand Soil with 15% FFA Addition

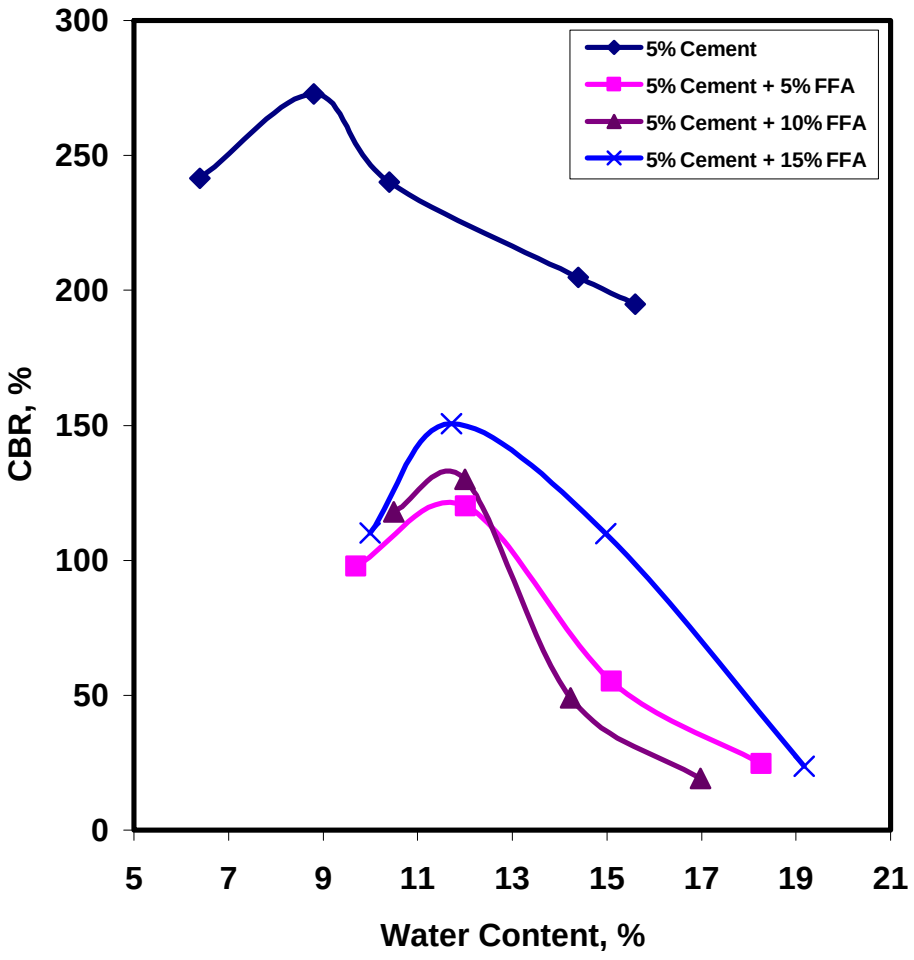

Figure 4-64: Effects of Moisture and 5% Cement with Different FFA Dosages on CBR of Sand

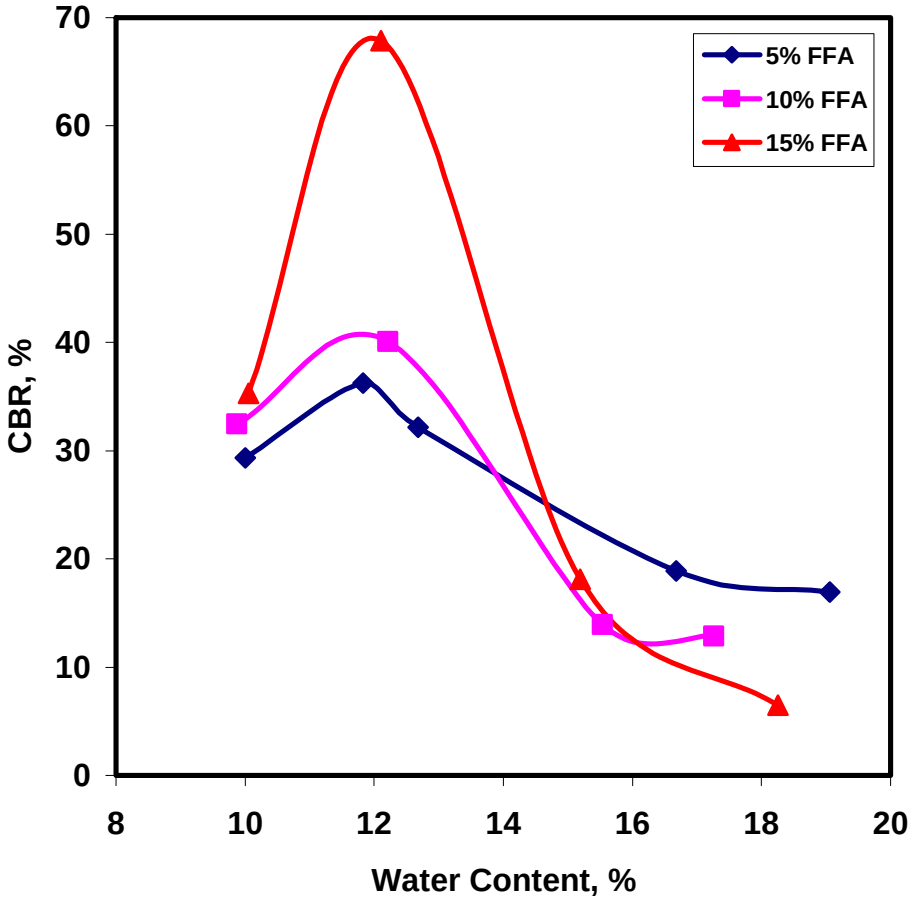

Figure 4-65: Effects of Moisture and Different FFA Dosages on CBR of Sand

Table 4-10: Compaction and CBR Test Results for FFA-Sand Soil

Cement (%)	FFA (%)	$(\gamma_d)_{max}$ kN/m^3	w_{opt} (%)	CBR (%)	w (%)
5	0	17.87	11	273	8.8
5	5	17.625	12	120	12
5	10	17.3	13.7	133	11.8
5	15	16.96	14.3	151	11.7
0	5	16.985	12.7	36	11.8
0	10	17.424	12.2	40	12
0	15	17.778	12.1	68	12.1

4.5.3 Unconfined Compressive Test Results

The unconfined compression test was adopted as a basic test to study the effect of age and additive content on strength gain of the treated sand mixes. In this test, all the samples were compacted at the optimum moisture content in a mold of h/d = 2 (d = 50, h = 100 mm). All the samples were tested in duplicate according to the ASTM D 2166 procedure. Only the sealed regime was adopted, and all samples were cured at the laboratory condition (23 ± 3°C).

4.5.3.1 CKD-Sand Mixtures

The sand soil was treated with different percentages of CKD and cement and CKD alone, as listed in Table 3.2. The effect of age and CKD content on the unconfined compressive strength gain is discussed thoroughly in the following sections.

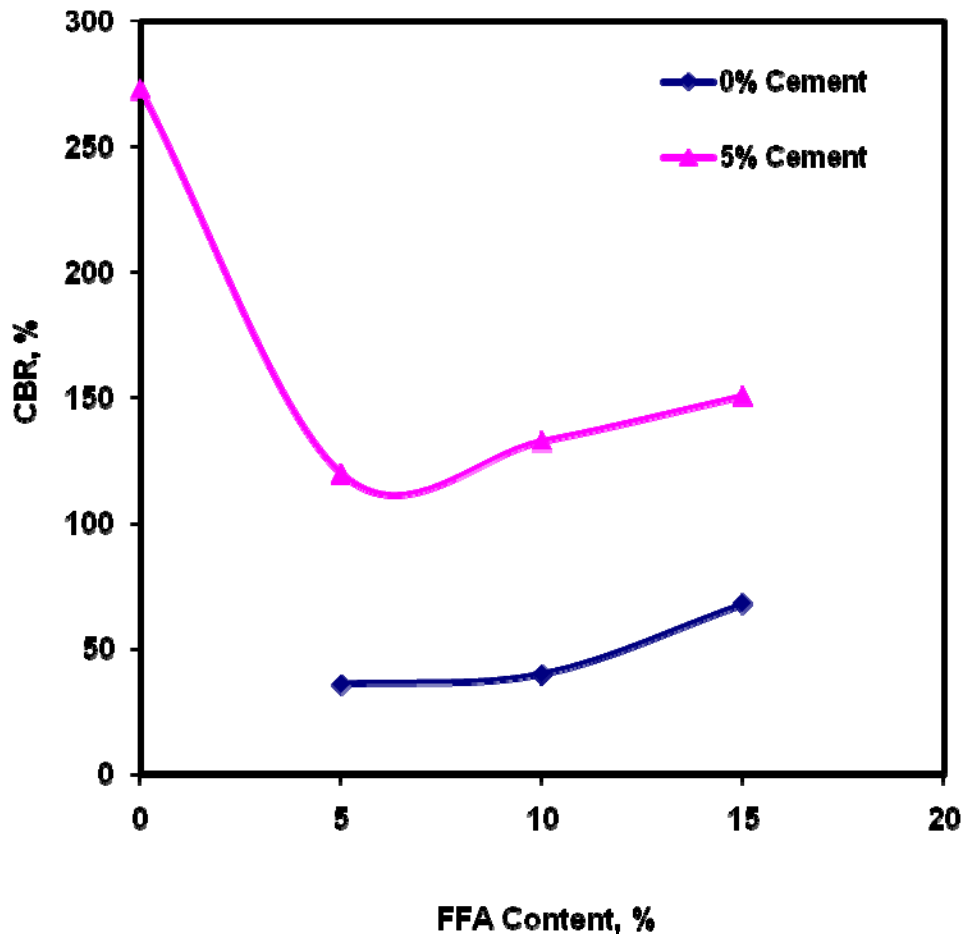

Figure 4-66: Maximum CBR Value-FFA Content Relationship for Sand Soil

a. Effect of curing time

All samples were prepared and tested after curing periods of 3, 7, 14, and 28 days at the laboratory temperature (23 ± 3°C). The variation of unconfined compressive strength (q_u) of CKD-sand mixtures with the curing period is presented in Figure 4.67 and Figure 4.68. The results in these figures clearly indicate that there was an approximately linear relationship between q_u and curing periods. Moreover, it can be seen that, as the curing period increased, the unconfined compressive strength increased, and there was a continued gaining strength with time. This continued gaining strength was mainly due to the availability of sufficient moisture content during the course of CKD hydration since the samples were sealed. The reason for the increase in strength is attributed to the fact that cement and CKD usually need some time to develop their strength and as the curing period increases, the cement-CKD hydration products will increase leading to an increase in strength.

b. Effect of additive content

Figure 4.69 and Figure 4.70 depict the relationship between the unconfined compressive strength (q_u) and CKD content. It is observed that the q_u increased sharply with increasing the CKD content for the samples with 2% cement (Figure 4.69) or without cement (Figure 4.70) in an approximately linear relationship. It is also noted from these two figures and Table 4.11 that the sand stabilized with combined stabilizer (CKD + cement) has developed higher strength than those stabilized with CKD only (i.e. the 7-day q_u was 755.03 kPa and 497.75 kPa for the sand treated with 2% cement plus 10% CKD and 10% CKD alone, respectively).

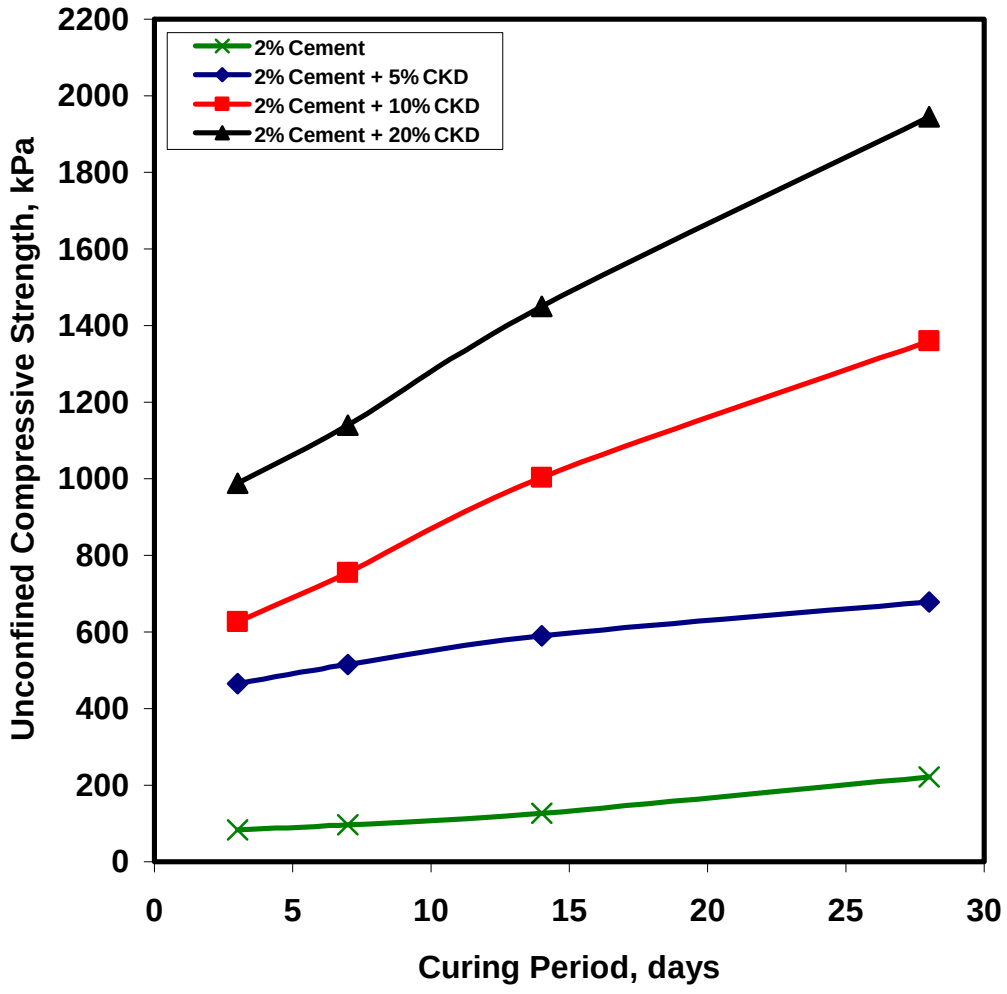

Figure 4-67: Variation of q_u with Curing Period for CKD-Cement-Sand Mixtures

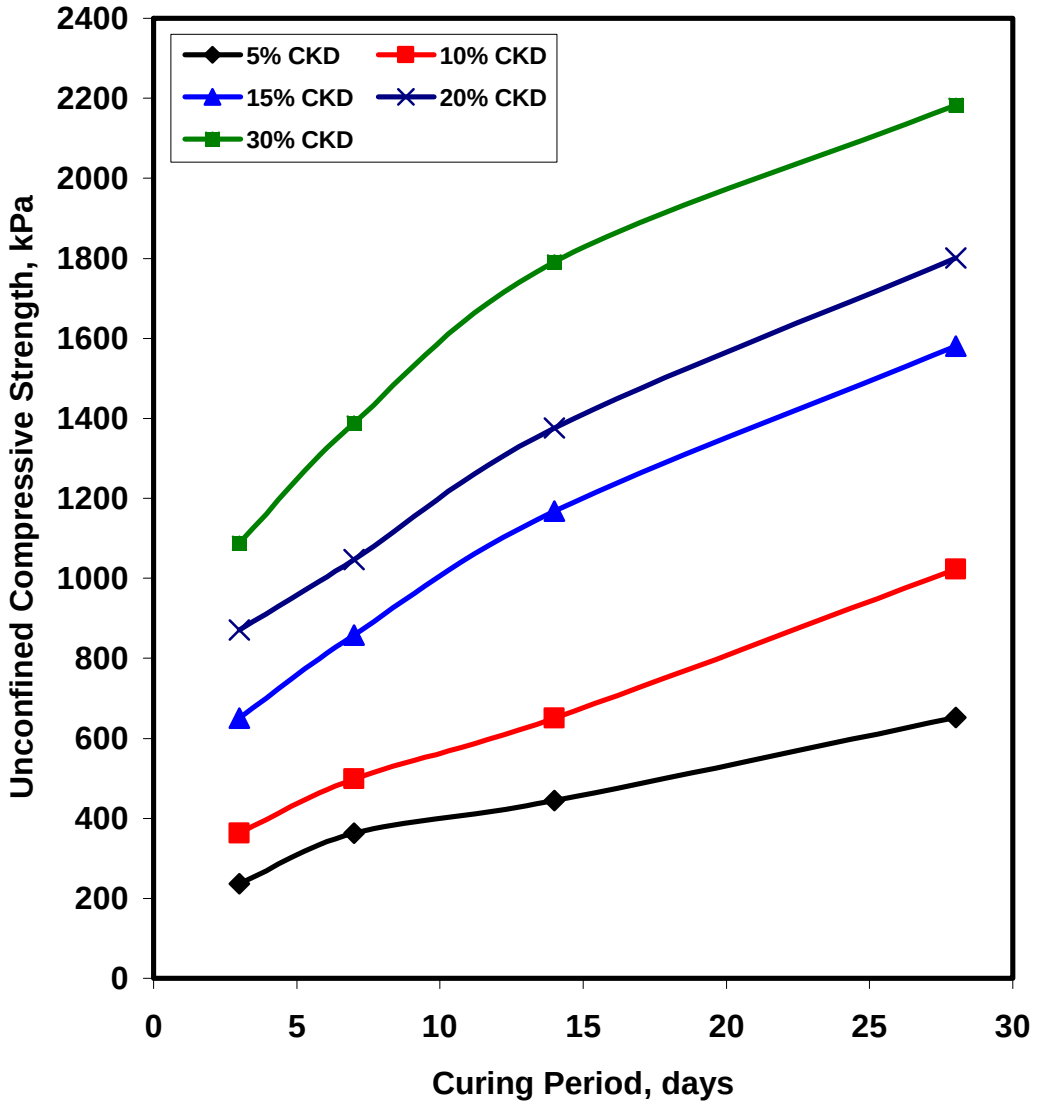

Figure 4-68: Variation of q_u with Curing Period for CKD-Sand Mixtures

According to the ACI Committee 230 Report (ACI, 1990), the minimum 7-day q_u specified for subbase and subgrade layers in rigid pavement construction by the USA Army Corps of Engineers (USACE) is 1,380 kPa (200 psi) and for base course 3,450 kPa (500 psi). For flexible pavement construction, however, these values are 1,725 kPa (250 psi) and 5,175 kPa (750 psi), respectively. Results summarized in Table 4.11 indicate that the sand mixes treated with 30% CKD satisfied the 7-day strength requirements according to the ACI Committee 230 Report (ACI, 1990).

Table 4-11: Unconfined Compressive Strength Test Results for CKD-Sand Soil

Cement (%)	CKD (%)	Unconfined Compressive Strength (kPa)			
		3 days	7 days	14 days	28 days
2	0	82.55	96.76	125.89	220.86
2	5	464.17	514.17	590.77	677.91
2	10	626.15	755.03	1003.30	1359.56
2	20	988.24	1139.83	1450.00	1945.43
0	5	236.92	362.64	444.06	652.47
0	10	362.64	497.75	649.56	1022.64
0	15	650.33	858.24	1167.58	1580.43
0	20	870.33	1046.76	1375.60	1801.20
0	30	1087.91	1388.89	1791.42	2183.06

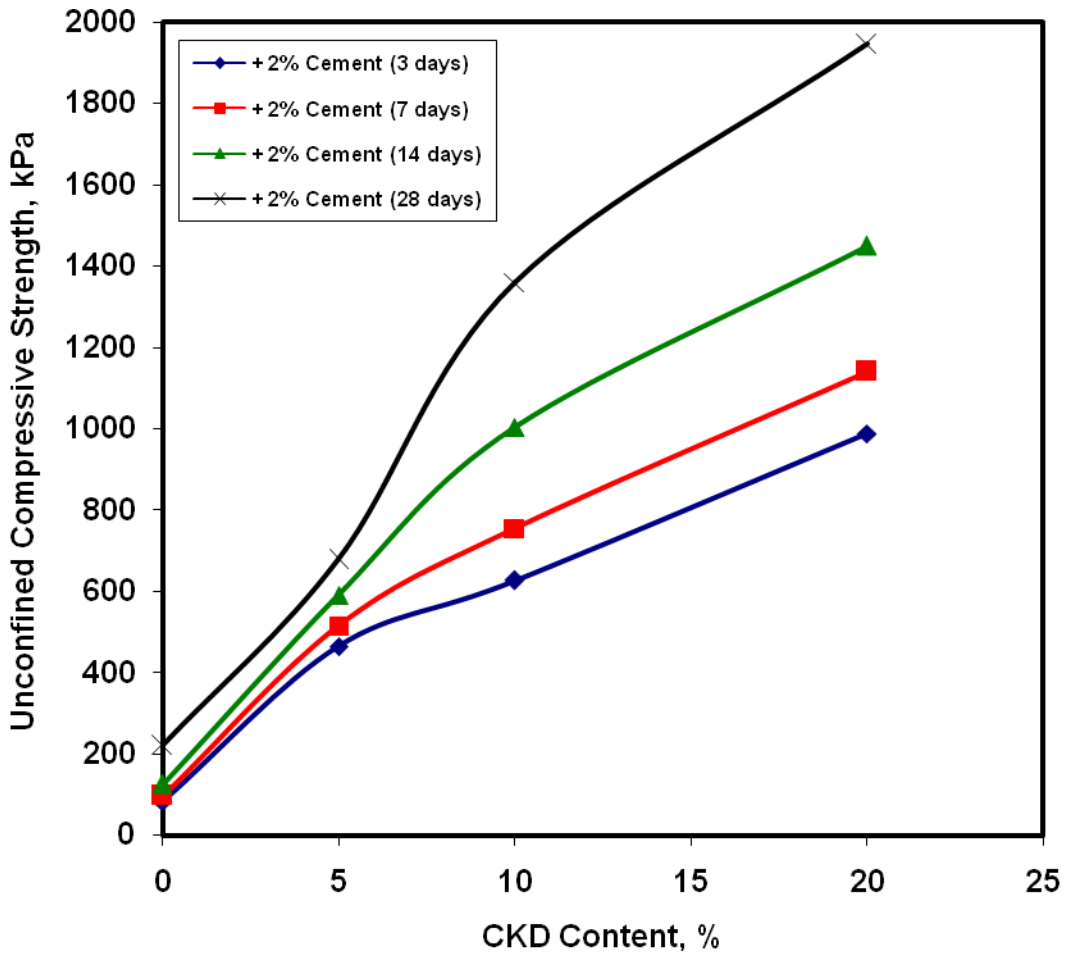

Figure 4-69: Variation of q_u with CKD Content for CKD-Cement (2%)-Sand Mixtures

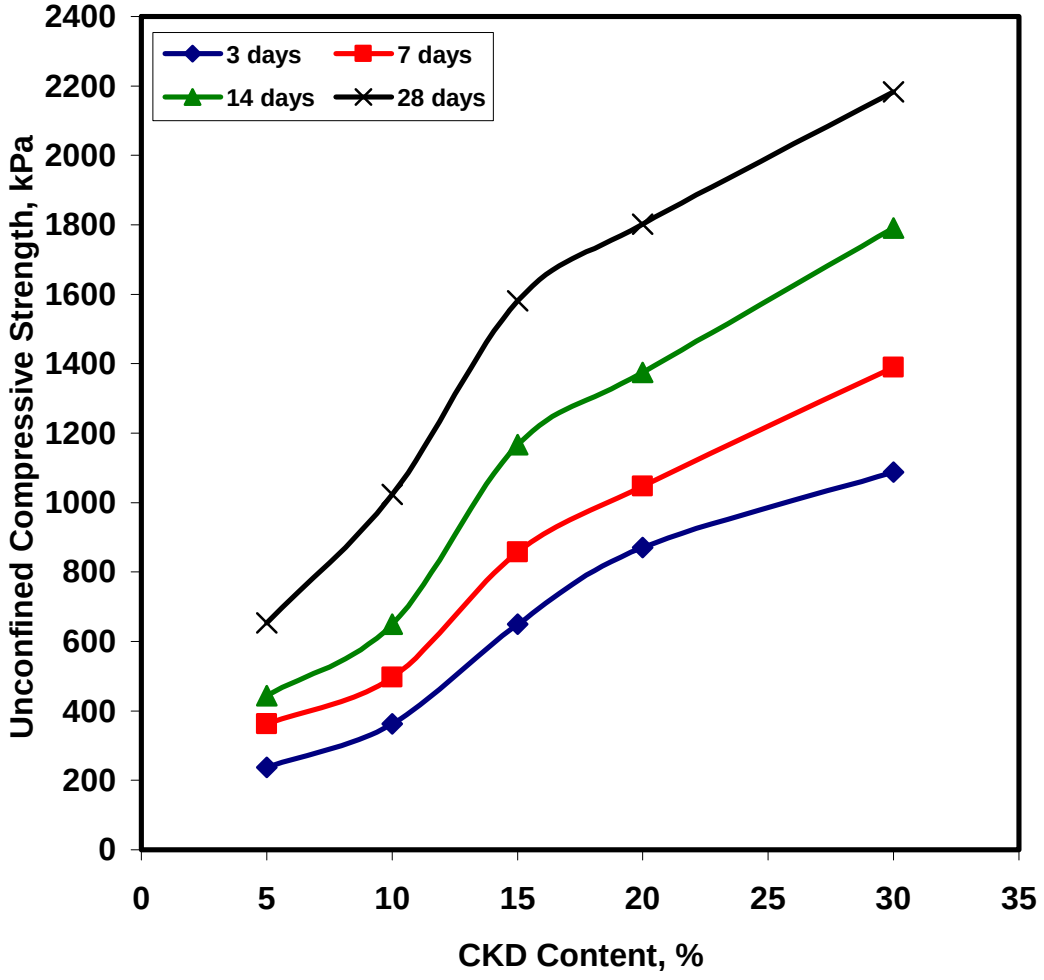

Figure 4-70: Variation of q_u with CKD Content for CKD-Sand Mixtures

4.5.3.2 FFA-Sand Mixtures

Various percentages of FFA with 5% cement or FFA alone, as listed in the Table 3.3 were used to treat the sand soil. All samples were compacted at the optimum moisture content in a mold of 50 mm diameter and 100 mm height (h/d = 2). Thereafter, the samples were sealed and left to cure at laboratory condition (23 ± 3°C) until testing. All samples were tested according to the ASTM D 2166 procedure. The effect of age and FFA content on the strength gain is discussed thoroughly in the following sections.

a) Effect of curing period

Sealed samples were prepared at the optimum moisture content and tested after curing periods of 3, 7, 14, and 28 days at the laboratory temperature (23 ± 3°C). The effect of curing period on strength of sand-FFA mixtures was studied.

The relationship between the unconfined compressive strength (q_u) and the curing period is depicted in Figures 4.71 and 4.72. The results in these two figures clearly indicate that the strength increased with the extended period of curing. It can be observed that the relationship between q_u and curing period was nearly linear and there was a continued strength gain during the whole period. This is attributed to the availability of sufficient moisture content for cement hydration (Figure 4.71) since the specimens are sealed. For the sand samples admixed with FFA (Figure 4.72), the rate of strength increase was marginal due to the absence of cement. The increase in strength in these specimens could be ascribed to the marginal loss of moisture.

Data in Figures 4.71 and 4.72 indicate that the rate of strength gain was higher in the initial days of curing and, thereafter, it began to decrease. From Figure 4.71 and Table 4.12, it is also noted that the addition of FFA to cement-stabilized sand reduced the

strength (i.e. the 28-day q_u for sand stabilized with 5% cement was reduced from about 701 kPa to about 305 kPa due to the addition of 5% FFA). Further, the 15% FFA samples had the highest strength as compared with the 5 and 10% FFA samples. The unconfined compressive strength of treated sand with FFA alone was very low, even with the 15% FFA addition.

b) Effect of additive content

Figure 4.73 and Figure 4.74 display the relationship between the unconfined compressive strength (q_u) and additive content for the FFA-cement-sand and FFA-sand mixtures**.** From the data in these two figures, it is noted that the addition of FFA to the cement stabilized sand reduced the q_u i.e. q_u after 7 days for 5% cement stabilized sand was reduced from 343 kPa to 135 kPa when 5% cement and 5% FFA additives were added. Thereafter, q_u began to increase with the increase in the FFA addition. However, the strength was still lower than those treated with only 5% cement (Figure 4.73). Such reduction indicates that the addition of FFA reduced the positive effect of the cement substantially. As a result, FFA additive is not a suitable stabilizer for sand soil.

Results clearly indicate that, although, the unconfined compressive strength increased with increasing the FFA content (Figure 4.74 and Table 4.12), the q_u values were still much lower than that stabilized with 5% cement only. The unconfined compressive strength of treated sand with FFA alone was very low, even with the 15% FFA addition. Results also indicate that none of the FFA-cement-sand or FFA-sand mixtures satisfied the 7-day strength requirements, according to the ACI Committee 230 Report (ACI, 1990). The FFA addition did not bring about significant improvement in strength.

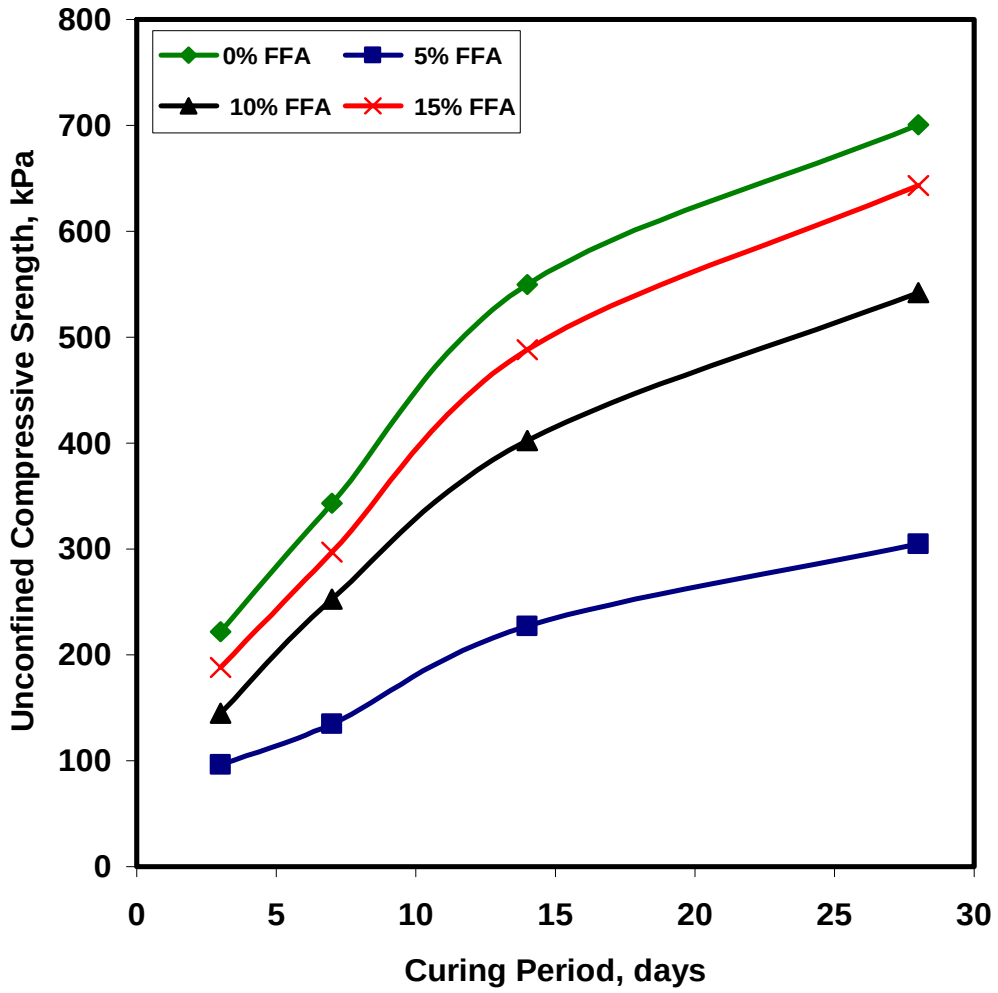

Figure 4-71: Variation of q_u with Curing Period for FFA-Cement (5%)-Sand Mixtures

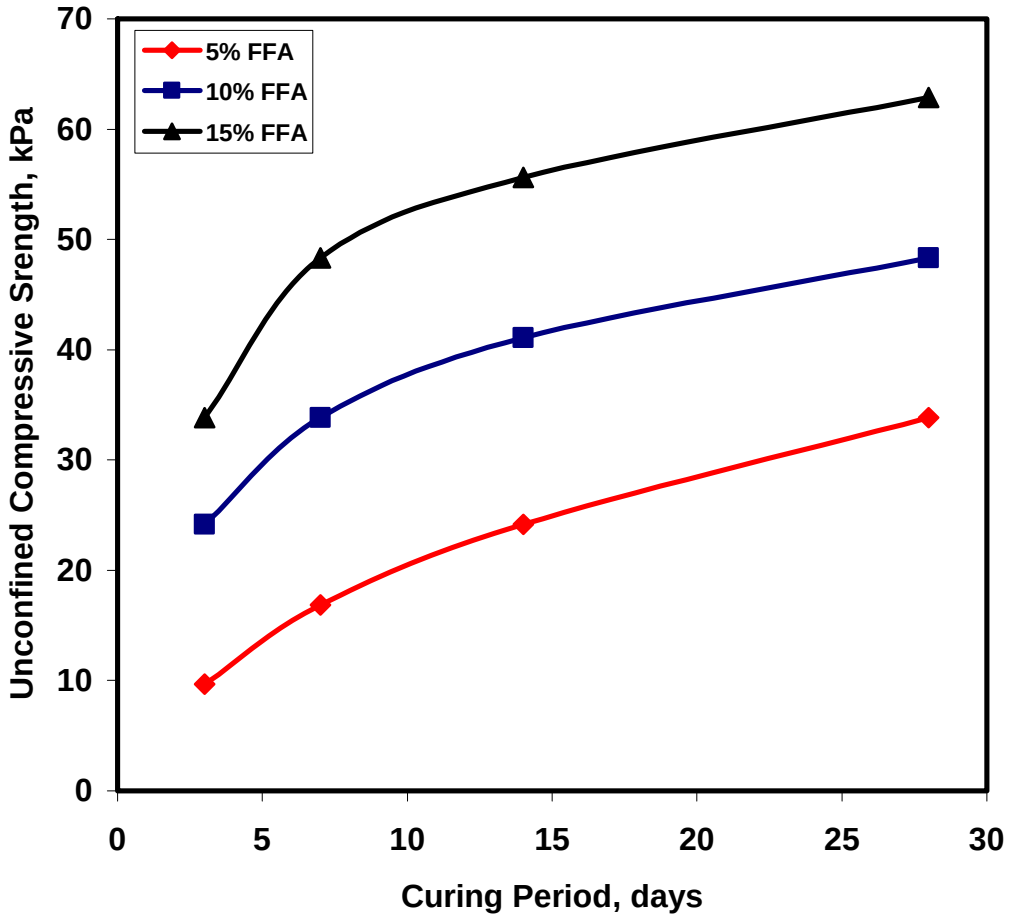

Figure 4-72: Variation of q_u with Curing Period for FFA-Sand Mixtures

Table 4-12: Unconfined Compressive Strength Test Results for FFA-Sand Soil

Cement (%)	FFA (%)	Unconfined Compressive Strength (kPa)			
		3 days	7 days	14 days	28 days
5	0	221.65	343.41	549.72	700.46
5	5	96.70	134.78	227.25	304.61
5	10	145.06	252.42	402.20	541.98
5	15	187.97	297.31	488.24	643.07
0	5	9.67	16.89	24.18	33.85
0	10	24.18	33.85	41.10	48.36
0	15	33.85	48.35	55.61	62.86

It is well known that cement is effective in treating sandy soils. On the other hand, FFA does not have inherent cementitious properties by itself, rather it produces cementitious material. As a result, FFA additive is not a suitable stabilizer for sand soil. The above preliminary results vividly indicate that CKD only succeeded in improving strength of sand-stabilized mixtures.

4.5.4 Durability (wetting and drying) Test

CKD-sand mixtures with or without cement, as listed in the Table 3.1, were subjected to the ASTM D 559 standard durability test and the modified slake durability test. It is worth mentioning that FFA-sand mixtures were excluded from the durability tests because they produced a low strength and failed to satisfy the 7-day strength requirements, according to the ACI Committee 230 Report (ACI, 1990).

Figure 4.75 and Figure 4.76 present the results obtained from the two durability tests. From the data in these two figures, it can be noted that as the CKD content increased, the weight loss decreased. The average weight losses of the all mixtures at the end of 12 cycles are summarized in Table 4.13. These results indicate that the maximum weight loss for the all mixtures, except for the 2% cement with 5% CKD mixture, did not exceed the maximum allowable weight loss of 14% as set forth by the Portland Cement Association (PCA) for cement-soil mixtures. Comparison of the data in Figures 4.75 and 4.76 and in Table 4.13 indicates that the weight loss obtained by the slake durability test was always more than that by the ASTM D 559 for all the samples tested for durability assessment. This finding is in agreement with results reported by Ahmad (1995).

Table 4-13: Weight Loss for CKD-Sand Mixtures after 12 cycles

CKD (%)	Cement (%)	Weight Loss (%)					
		ASTM D 559			Slake Durability		
		Sample # 1	Sample # 2	Average	Sample # 1	Sample # 2	Average
5	2	34.3	36.9	35.6	50.9	50.6	50.8
10	2	2.1	2	2	3	3.7	3.4
20	2	1.6	1.8	1.7	2.2	3.1	2.6
10	0	7.7	7.2	7.5	10	11.7	10.8
20	0	3.2	3.6	3.4	4.3	5.5	4.9
30	0	2.7	2.5	2.6	4.3	3.5	3.9

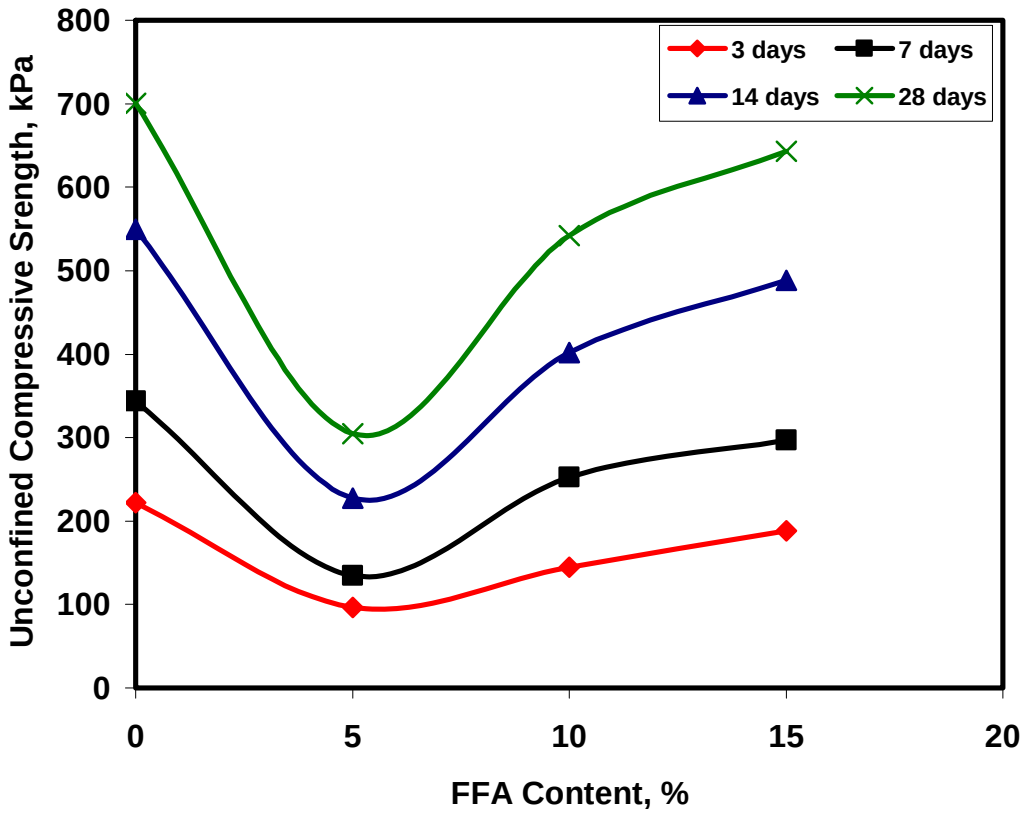

Figure 4-73: Variation of q_u with FFA Content for FFA-Cement (5%)-Sand Mixtures

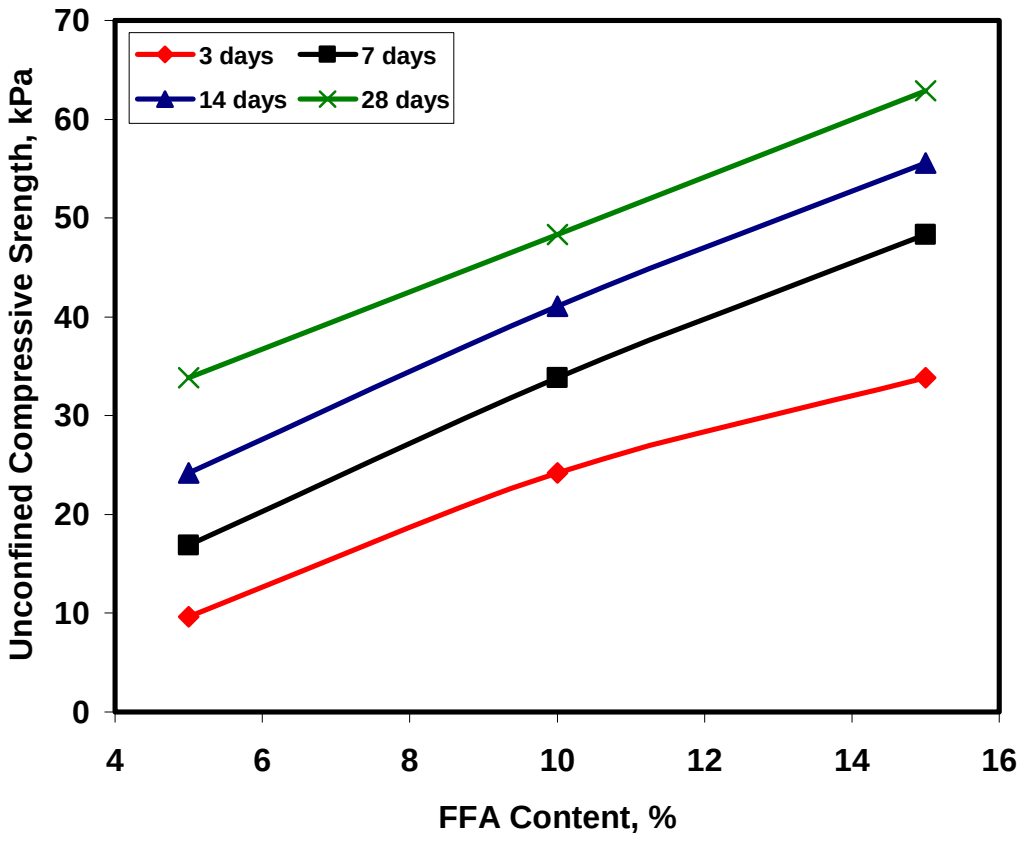

Figure 4-74: Variation of q_u with FFA Content for FFA-Sand Mixtures

The data in the Table 4.13 above indicate that the weight loss was lower when 2% cement was used with all the CKD additions in sand treatment. Furthermore, the weight loss was very high when 2% cement and 5% CKD additions were used. However, the weight loss reduced sharply with the addition of 2% cement and 10% CKD to the sand soil. According to the durability results and strength requirements, only sand stabilized with 30% CKD is suitable for base and subase layers. However, the other mixtures (10% and 20% CKD) can be utilized in other field work like improving the bearing capacity of sand and marl soils of low to moderate high rise building and pavements.

4.6 Economy

It is important to mention that CKD is generated at approximately 30 million tons worldwide per year [maslehuddin et al., 2008]. Modern dust-collecting equipment is designed to capture virtually all CKD and much of this material can today be returned to the kiln, for various reasons, a significant portion, in some cases as much as 30 to 50% of the captured dust, must be removed as an industrial waste [Kessler, 1995; USEPA, 1998]. Since CKD is considered a waste material and such a significant percentage (30 to 50%) must be removed as an industrial waste material, it is economical and very beneficial to use this material in many engineering applications such as soil stabilization. In addition, comparison the cost of cement (very expensive) with that of CKD (very cheap) indicates that it is economical to use CKD instead of cement in soil stabilization.

Similarly, FFA is considered a waste material and very cheap compared with cement. However, FFA may have hazardous ingredients that are deleterious to the ground water and environment. Therefore, caution has to be practice to warranty that there will be no bad impact on the environment. The following section will show that clearly.

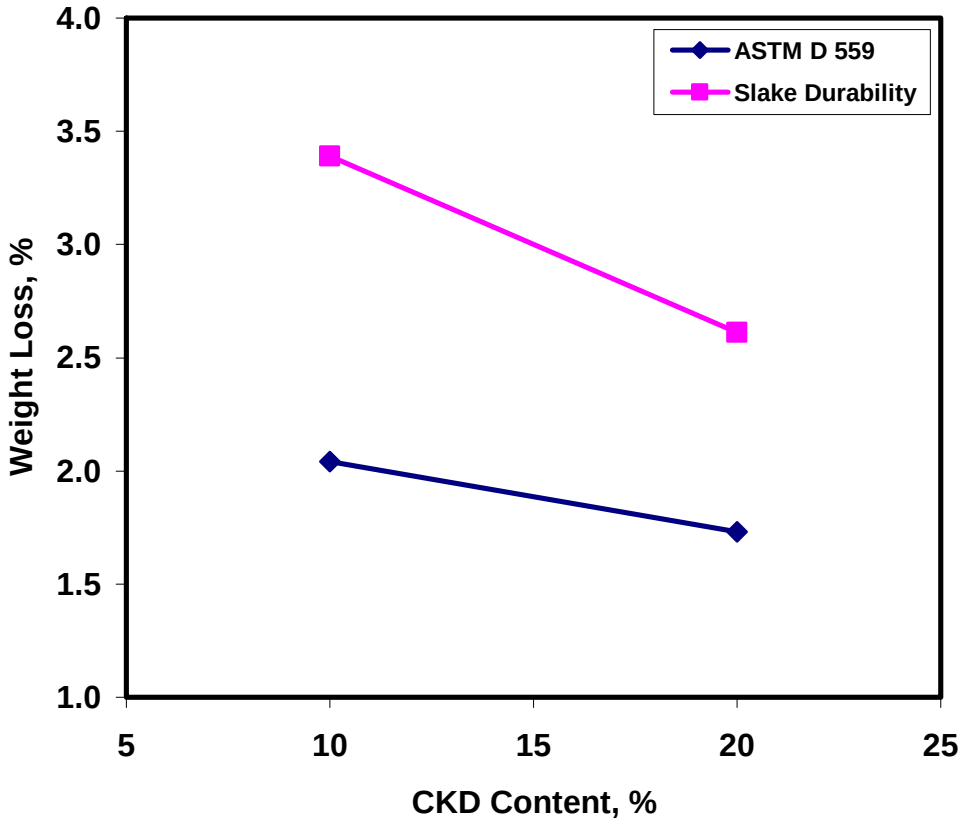

Figure 4-75: Variation of the Weight Loss with CKD Content and 2% Cement for Sand Soil

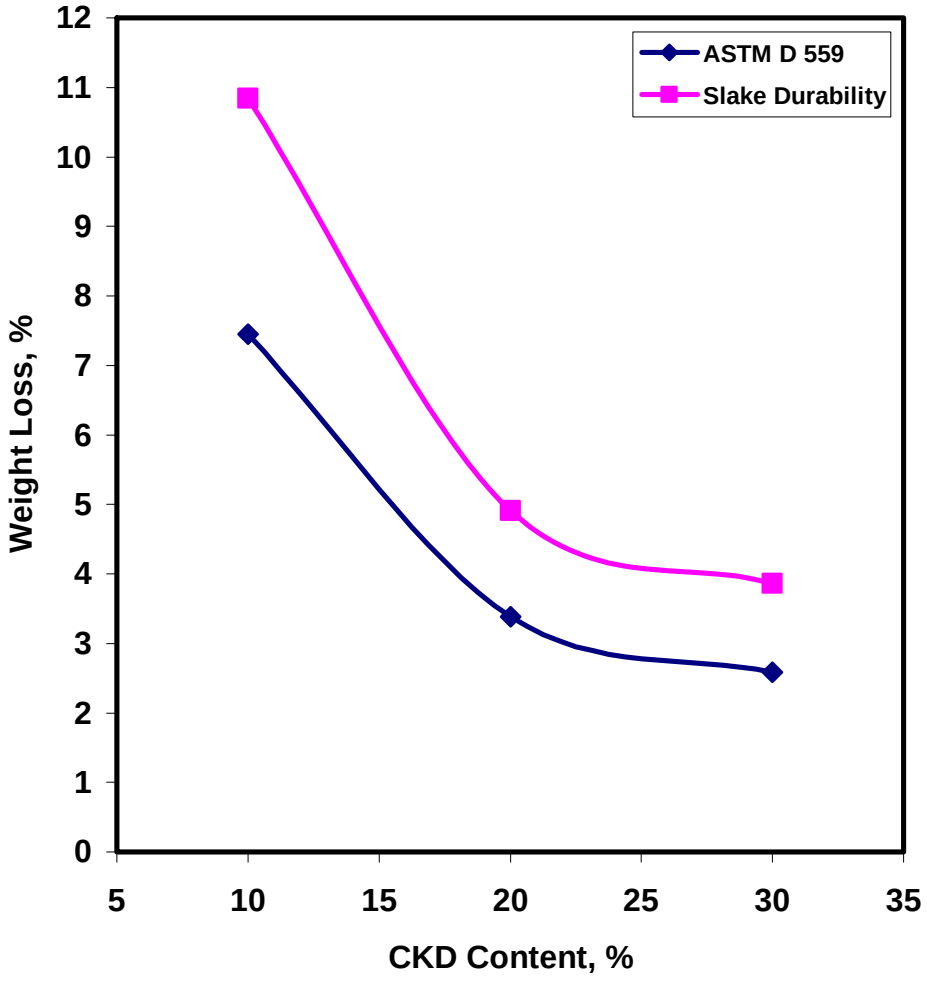

Figure 4-76: Variation of the Weight Loss with CKD Content for Sand Soil

4.7 Toxicity Characteristic Leaching Procedure (TCLP) Results

The TCLP set by the United States Environmental Protection Agency (USEPA) was performed on two specimens of non-plastic marl soil stabilized with 5% cement plus 5% FFA which have already satisfied the strength requirements and durability assessment. These two specimens were prepared at the Geotechnical Laboratory and made ready for the TCLP tests that were conducted at the Center of Environment and Water, Research Institute.

The eight USEPA-regulated TCLP metals are arsenic, barium, cadmium, chromium, lead, mercury, selenium and silver. The concentrations of the regulated-metals that leached from the stabilized soil samples are shown in Table 4.14 and compared with the maximum concentrations set by the USEPA for toxicity characteristics of the regulated metals. The Table clearly shows that all the concentrations of the leached metals are below the USEPA maximum concentration for toxicity characteristic. However, the concentrations of vanadium and nickel in the TCLP extraction are found to be relatively high despite the fact that both of these metals are not regulated by the EPA. However, the maximum allowable concentration of vanadium in the drinking water for farm animals is 0.1 mg/l and for irrigation, it is about 0.1 to 1.0 mg/l. Similarly, the allowable level of nickel in groundwater for irrigation varies between 0.2 and 2.0 mg/l [Hadi, 1993].

The exposure to high levels of vanadium can cause harmful health effects such as lung irritation, coughing, wheezing, chest pain, runny nose, and a sore throat. The Occupational Safety and Health Administration (OSHA) has set an exposure limit of 0.05 milligrams per cubic meter in air (0.05 mg/m^3) for vanadium pentoxide dust and 0.1

mg/m^3 for vanadium pentoxide fumes [ATSDR, 1992]. The National Institute for Occupational Safety and Health (NIOSH) has recommended that 35 mg/m^3 in air of vanadium be considered immediately dangerous to life and health. This is the exposure level of a chemical that is likely to cause permanent health problems or death [ATSDR, 1992]. From the previous discussion, it can be concluded that the usage of 5% FFA plus 5% cement in non-plastic marl soil stabilization is not suitable from health and environmental effects points of view. However, the use of 5% of FFA plus 5% of cement in soil stabilization satisfied the strength requirement and durability assessment.

Table 4-14: TCLP for Marl Soil Stabilized with 5% Cement and 5% FFA.

Metal	**EPA (mg/l)**	**Stabilized Marl Soil (5% Cement + 5% FFA)**	
		Sample # 1 (mg/l)	**Sample # 2 (mg/l)**
Ag	5	3.96	4.08
As	5	< 0.015	< 0.015
Ba	100	0.136	0.141
Cd	1	< 0.002	< 0.002
Cr	5	0.029	0.071
Hg	0.2	< 0.002	< 0.002
Pb	5	< 0.01	< 0.01
Se	1	< 0.02	< 0.02
Ni	NR	8.78	9.05
V	NR	16.2	16.2

NR: not regulated by EPA

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

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5.1 Summary

This research was done to stabilize two eastern Saudi soils, namely, non-plastic marl and sand. CKD and FFA were initially used to improve the quality of these soils. Several tests, including specific gravity, plasticity, grain-size distribution, compaction, CBR, unconfined compression, and durability, were performed to assess the engineering properties of plain soil (without stabilizer) and CKD- or FFA-treated marl or sand mixtures. CKD stabilization was found to be the most suitable and economical way of utilizing both non-plastic marl and sand soils. However, FFA was found to be suitable for the non-plastic marl only.

5.2 Conclusion

Based on the interpretation and analysis of the results presented in this research, the following main conclusions can be drawn:

- CKD was noted to be better than FFA for both soils in terms of strength and durability.
- FFA was found to be a suitable chemical addition to treat non-plastic marl soil. On the other hand, FFA did not bring about a significant improvement to sand soil in terms of strength.
- A CKD content of 30% was found to be adequate for the effective stabilization of sand soil. It met the strength and durability requirements.

- A CKD content of 20% plus 2% cement was found to be adequate for the effective stabilization of non-plastic marl. It met the strength and durability requirements.
- A FFA content of 5% plus 5% cement was found to be adequate for the effective stabilization of non-plastic marl. It met the strength and durability requirements. However, the vanadium and nickel content is high and may result in health hazard.
- There was a continuous increase in the unconfined compressive strength with the increase in the curing period.
- There was a good consistency between slake durability and ASTM D 559. However, the weight loss obtained by the slake durability test was always more than that by the ASTM D 559. The consistency and accuracy of performing the slake durability tests are of great advantages over the ASTM D 559 test.
- As the CKD content increased, the weight loss decreased, indicating that CKD is a suitable stabilizing agent, not only from strength perspective but also from durability point of view.

5.3 Recommendations for Future Research

- To study the possibility of stabilizing other eastern Saudi soils, namely, plastic-marl, clay and sabkha using cement kiln dust and heavy fuel oil fly ash.
- To use higher dosages of CKD for the soils investigated.
- To perform the permeability test on the plain (untreated) and stabilized soils.
- The environmental impact should be examined before using the additives.

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